May 2014

90% Design Report Deadman's Basin Diversion Dam and Headgate Replacement Project

Prepared for:
Montana Department of Natural Resources and Conservation
State Water Projects Bureau
P.O. Box 201601
Helena, MT 59620-1601

Prepared By:
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Missoula, MT 59808

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1 INTRODUCTION

The Montana Department of Natural Resources and Conservation (DNRC) State Water Projects Bureau (SWPB) and the Deadman's Basin Water User's Association (DBWUA) selected Morrison-Maierle, Inc. (MMI) to perform the Engineering Design and Construction Services for the Deadman's Basin Diversion Dam and Headgate Replacement Project. This project will replace the existing diversion dam and headgates which supply water to the Deadman's Basin Project.

1.1 Project Background

The Deadman's Basin Water Project (DBWP) is located in Wheatland County Montana near the town of Shawmut, and is owned by the Montana Department of Natural Resources (DNRC) and is operated by the Deadman's Basin Water User's Association (DBWUA). Figure 1-1 presents the location of the Diversion Dam and Headgates.

Figure 1-1 Deadman's Basin Diversion Dam and Headgate Location

Originally constructed in 1941, the DBWP was created by a collaborative effort between the Work Projects Administration (WPA) and the Montana Water Conservation Board. Overall, the DBWP includes a 76,900 acre-foot off-stream reservoir, an 11.5 mile long, 600 cfs supply canal, and two delivery canals with a total length of approximately 12.5 miles. The DBWP serves over 160 farm and ranch families as well as the communities of Ryegate, Lavina, Roundup, Musselshell, and Melstone spanning over 200 river miles. This facility is a key resource for the economic livelihood of this area.

The DBWP is primarily supplied by diverting flows from the Musselshell River at the Deadman's Basin Diversion Dam. This diversion is located approximately 6 miles north-west of the town of Shawmut, MT and approximately 10 miles east of the town of Harlowton, MT, just south of Montana Highway 12. The existing diversion dam consists of a 6-foot tall by 222-foot long concrete weir wall sitting on top of a 10-foot long concrete apron with 4-foot deep cut-off walls at each side. The dam is in very poor condition and is near failure. Much of this damage occurred during 2011 flooding which resulted in most of the Musselshell River Corridor being designated a Federal Disaster Area by the Federal Emergency Management Agency (FEMA). The 2011 damage to the dam included major spalling, cracking, settlement, and scour extending underneath the concrete apron. Furthermore, riprap placed around the diversion after flooding in 1997 was completely missing.

In addition to the poor condition of the diversion dam, it also utilizes a less than ideal design. While effective for raising the upstream water surface elevation, the concrete weir wall diversion impacts the natural functions of the river and creates a potential safety hazard. The diversion impacts river function primarily by creating a fish barrier that significantly decreases the ability of fish to move upstream. Also, these types of diversions can be safety hazards because they tend to create dangerous hydraulic conditions downstream of the dam that once entered, is difficult for a person to escape. The Deadman's Basin Reservoir is a popular attraction for fishing and outdoor enthusiasts. Improving this diversion structure will improve fisheries and public safety, thus reducing the liability to the owners.

Water from the Musselshell River is directed into the DBWP supply canal through an existing headgate structure adjacent to the diversion dam. This existing headgate structure is composed of an approximately 30-foot wide by 13-foot tall rectangular concrete chute structure with two 14.5-foot wide radial gates. These headgates are more than 70-years old, are very difficult to operate, and leak excessively. They have been battered by flooding over the years, and are at the end of their useful life cycle. Replacement of these headgates would improve operation and increase efficiency by preserving approximately 3,000 acre-feet of water per year in the Musselshell River that is currently uncontrolled leakage into the DBWP supply canal.

After the damage to the Deadman's Basin Diversion Dam and Headgates in 2011, representatives from FEMA visited the site to assess its condition. DNRC also completed an RRGL Grant Application to replace the headgates, and contracted with an engineering consultant to conduct a Field Investigation, Preliminary Engineering Report, and complete a RDGP Grant Application to replace the diversion.

The DNRC's and DBWUA's efforts culminated in funding from FEMA, RRGL, and RDGP to replace the headgates and diversion dam. This is in addition to funding and in-kind services being committed by the DNRC, SWPB, and DBWUA. The existing reinforced concrete diversion dam will be replaced by a combination reinforced concrete and rock ramp diversion structure.

1.2 Project Approach

Morrison-Maierle has reviewed the existing information related to the diversion, including the Deadman's Basin Diversion Engineering Study Field Investigation Report, the Deadman's Basin Diversion Dam Engineering Study Perliminary Engineering Report, and the Reclamation and Development Grants Program RDGP Application for Deadman's Basin Diversion Dam.

Morrison-Maierle personnel also attended a project kick-off meeting at the site with project stakeholders to better define project goals and deliverables; and discuss design and construction constraints including project permiting, fish passage and Montana Fish Wildlife and Parks concerns, site dewatering, headgate automation, and the availability of other project data.

In addition, Morrison-Maierle personnel have had subsequent conversations with DBWUA personnel regarding current operational issues, and to discuss potential improvement options to address these issues.

Our approach to the project design has incorporated the available project data with input provided by project stakeholders to develop a project that will meet the long-term operational needs of DBWUA. First and foremost, the design needs to provide for the primary intended function of the project to deliver irrigation water while addressing improved public safety. The design also needs to provide improvements related to water control and headgate operation. Finally, the design also strives to balance additional operational issues related to the primary use of the project, including sediment transport and fish passage within practical budget constraints.

For the purposes of design methodology, we have separated the project into two distinct, but inter-related, components. These individual components include the headgate design and the diversion dam design as discussed below.

1.2.1 Headgates

The proposed headgate improvement involves removing the existing radial gates and extending the existing headworks structure approximately 15 feet to provide room for the construction of four 6-foot by 6-foot slide gates. The new slide gates will be placed downstream of the existing headworks with baffle walls. The primary efforts for the headgate design involve structural and hydraulic analysis.

1.2.2 Diversion Dam

The proposed diversion dam improvement involves providing for the stabilization and repair of the existing diversion dam, and constructing a rock ramp structure adjacent to the downstream face to improve diversion hydraulics. The rock ramp will include a low flow/fishway/boater passage channel, as well as a sediment sluiceway. The list below presents the individual components necessary for this design.

<u>Rock Ramp Design Components:</u>

- Geomorphic Interactions

- Geometry and Hydraulics
- Riprap Design
- Sediment Transport
- Fish Passage Criteria
- Structural Design

2 HEADGATE DESIGN

The proposed headgates and associated headworks structure have been designed to incorporate features intended to mitigate for and improve upon current operational and maintenance issues faced by DBWUA personnel with respect to the existing headgates. In summary, these issues include the following.

- Excessive leakage related to the existing radial gates.
- Difficulty of gate operation, especially during icing conditions.
- Limited access to the fronts of the gates for debris removal, especially during high flow conditions.

To address these issues, the existing radial gates will be removed and replaced with a set of four 6-foot by 6-foot slide gates set into a new headwall in the canal immediately downstream of the existing headworks. The existing concrete headworks are being retained as they will provide significant protection to the new headgates from large floating debris impacts during high water events.

The slide gates will be supplied by a known and reputable manufacturer, and specified to a meet or exceed a typical industry leakage rate of less than 0.1 gallons per minute per linear foot of peripheral foot of gate. For a fully submerged 6-foot by 6-foot gate, this equates to approximately 2.4 gallons per minute per gate.

The high quality gates specified will also greatly improve ease of operation as the threaded risers allow for both an opening and closing force to be applied to the gates, as opposed to being able to apply only an opening force on the current radial gates. This will improve the ability of the gates to be fully closed under more challenging icing conditions. This can help avoid the situation that occurred earlier this spring where a radial gate was stuck open through a rapid rise in river stage, leading to downstream canal capacity issues.

The downstream location of the new gates will also provide improved equipment access to the upstream and downstream gate faces during high flow conditions, allowing for better removal of debris or ice inhibiting the function of the gates.

2.1 Headgate Options

Several different suppliers fabricate headgates that meet the size requirements of this project. The principal suppliers that both manufacture gates of sufficient size, and produce a product of sufficient quality include HydroGate and Waterman Industries.

Waterman Industries produces a high quality self contained slide gate that can be specified in aluminum, stainless steel, cast iron, or carbon steel. The aluminum or stainless steel versions will provide the best long term performance in this situation, but carbon steel will also perform well.

HydroGate produces a very high quality self contained stainless steel slide gate, and is often used in industrial applications. Their gate will perform similarly to the Waterman slide gate with respect to anticipated leakage rates, and will likely perform somewhat better with respect to gate rigidity and structural integrity. Their carbon steel gate option will also perform well under the anticipated project conditions at a lower price, but with reduced long-term corrosion resistance. HydroGate also produces an aluminum gate, but due to the potential for significant icing conditions at this location, they recommend using their steel gate options to better accommodate these conditions.

Gate pricing varies significantly depending upon material type and manufacturer. Preliminary pricing estimates from both Waterman and HydroGate suggest a per gate price range of approximately $13,000 to $21,600, with HydroGates stainless steel option being the most costly. These cost numbers are for procurement only, and do not include the cost of installation. Additional information related to each headgate manufacturer’s product line is presented in Appendix B.

Headgate lead times from placement of order to delivery can be quite long. HydroGate reports a 16-week lead time from placement of order to delivery, and Waterman reports a minimum 8-week lead time from placement of order. Past experience with Waterman in particular suggests that their lead time could be closer to a 12 to 16-week timeframe. Due to the likelihood of a long lead time for headgate delivery, we recommend that the gates be procured separately from the project bid process.

To this end, a gate procurement specification and bid schedule has been developed specific to the needs of the project. These documents provide minimum performance specifications, and request pricing for each of the three material options (aluminum, stainless steel, carbon steel). Early procurement will benefit the overall construction schedule, and provide the DNRC and DBWUA with accurate cost data to make an informed selection of gate material type.

2.2 Headgate Hydraulics

A rating curve was developed for both the existing and proposed headgates using HEC-RAS Ver. 4.1.0. This model spans the supply canal from upstream of the existing headgates, to approximately 3,700-feet downstream of the headgates. The model includes the existing siphon under Highway 12, and the culvert crossing approximately 900-feet downstream of the siphon. This model was created using existing survey data collected by several sources and is on NAVD 88 datum. The flow model was calibrated using observations obtained by the DBWUA Manager, Teri Hice. The rating curves calculated by this flow model for the existing and proposed headgates are shown in Figures 2-1 and 2-2. Appendix A presents additional information from the flow model, including a drawing of the cross section locations, and water surface profiles.

Figure 2-1 presents a 600-cfs capacity for the existing headgates when the headwater is at the diversion dam crest with only one gate open. Figure 2-2 presents the proposed headgates to have slightly less than 600-cfs capacity when the headwater is at the diversion dam crest with only 2 gates open. These results seem reasonable since the proposed headgate structure has an additional concrete peir wall between the two open headgates which does not exist for the existing headgates. The resulting rating curves also show both the proposed and existing headgates to have more than adequate capacity when all gate openings are utilized.

Figure 2-1 Existing Headgates Rating Curve

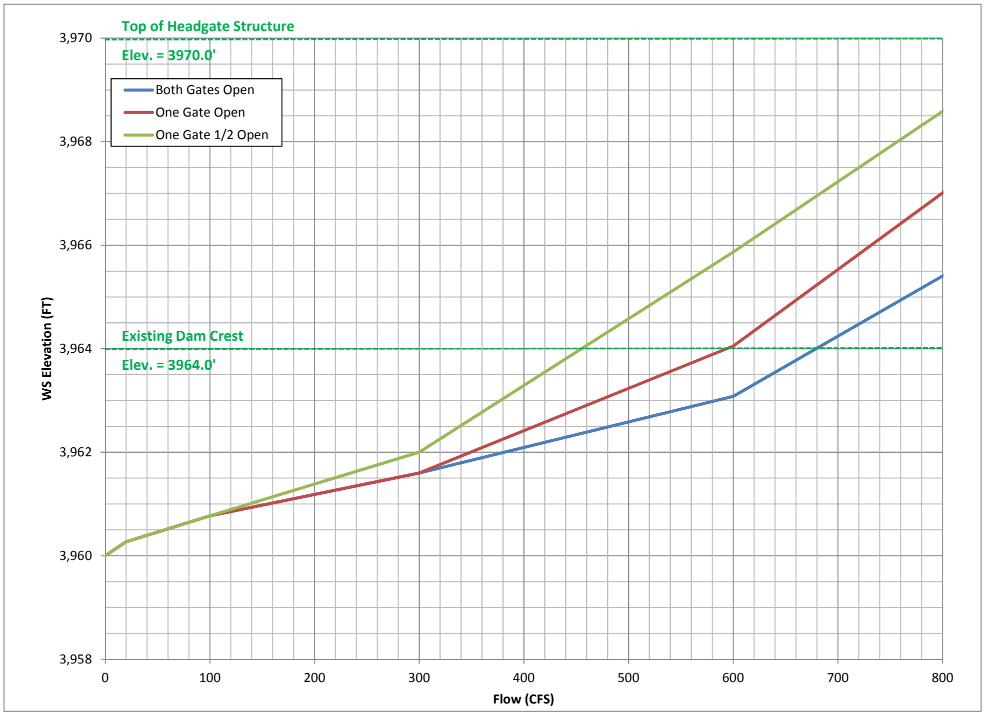
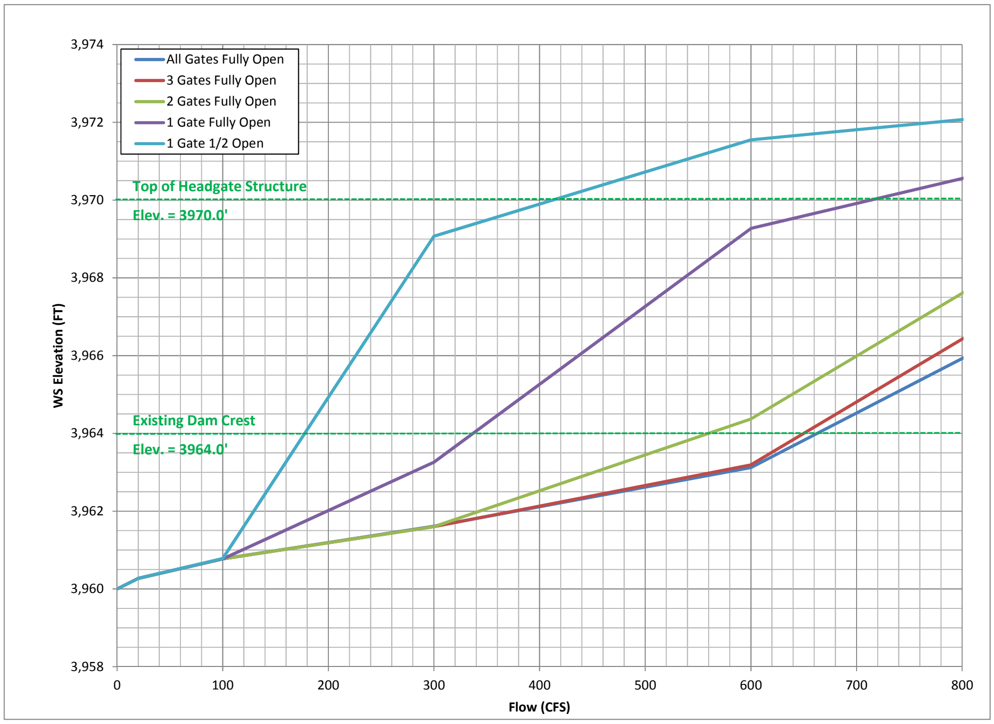


Figure 2-2 Proposed Headgates Rating Curve



2.3 Headgate Structural Design

The structural design effort for the headgate structure has been completed as a part of this phase of the project. The structure has been designed per 2009 International Building Code, ASCE 7-05 Minimum Design Loads for Buildings and Other Structures, ACI 318-08 Building Code Requirements for Concrete Structures, and American Welding Society D1.1-04 "Structural Welding Code". Specific design loads are provided on the drawings.

The structural design effort also included details and specifications for the walkway and railing above the slide gates.

2.4 Outlet Erosion Protection

Erosion protection at the outlet of the headgate structure is especially important since there is an inverted siphon a short distance downstream of the headgates. Any material eroded by the headgates will likely be deposited in the siphon. The United States Bureau of Reclamation's (USBR) "*Design of Small Canal Structures*" was used to design the riprap erosion control downstream of the headgate structure.

The guidance in section 7-12 of this manual recommends riprap erosion control to be placed at a length of four times the normal depth. The calculated normal depth at 600-cfs in the canal at the downstream end of the proposed headgate structure is just less than 5-feet. Using this criteria results in a required riprap length of 20-feet. The criteria also specifies that either an 18-inch or 12-inch riprap sizing be used. The riprap used for the rock ramp has a D_{50} of 30-inches, however, the material remaining after the rock ramp is completed will likely suffice to be used for the erosion control. The riprap erosion control downstream of the headgates will be specified at a minimum thickness of 24-inches, with a geotextile filter fabric along the subgrade.

3 DIVERSION DAM DESIGN

The proposed diversion dam replacement involves removing portions of the existing diversion dam and constructing a rock ramp structure in its place. The rock ramp includes both free-placed and grouted riprap. The rock ramp will include a low flow/fishway/boater passage channel, as well as a sediment sluiceway. The list below presents the individual components necessary for this design. Each of these design components are also discussed in some detail.

Rock Ramp Design Components:

- Geomorphic Interactions
- Geometry and Hydraulics
- Riprap Design
- Sediment Transport
- Fish Passage Criteria
- Structural Design

3.1 Local and System Interactions

Given recent flood events, the design of the rock ramp diversion structure needs to account for the fact that the Musselshell River experiences long- and short-term changes on a local and widely distributed spatial scale.

The existing structure has performed adequately over the last 70 plus years for its primary intended purpose of facilitating irrigation deliveries, and this long-term performance is being used as a valuable guide in our efforts to predict the future performance of the proposed design.

Priorities, design methodologies, and our general understanding of river mechanics have changed greatly between the time when the original structure was designed and constructed to today. Potential long-term issues related to channel scour and short-term issues related to sediment transport during high flow events were likely not given a high priority in the design process.

Localized changes due to significant scour induced channel down-cutting immediately downstream of the diversion have adversely affected the stability of the structure. This is in part due to the inability of the existing structure to effectively pass sediment at channel-forming flow events. This has created a change in local bed elevation that has altered the original design parameters of the structure, and is leading to project failure.

Our understanding of these river process and their potential affects related to upstream channel aggradation, downstream channel degradation, and lateral channel migration are a high priority in this effort, and have guided our design process. This understanding has primarily driven our design approach related to providing for improved sediment passage at the diversion structure, and improving flow hydraulics over the diversion structure to mitigate for and reduce long-term downstream bed scour.

Our design approach does maintain the existing diversion crest alignment for practical considerations as this alignment has created no significant adverse operational or geomorphic issues over the last 70 plus years of operation.

3.2 Geometry and Hydraulics

The geometry of the proposed rock ramp diversion is designed to adequately meet project goals and to be cost efficient with respect to use of materials. The new diversion follows the existing alignment and crest elevation to maintain a similar upstream water surface elevation. The layout of the rock ramp structure includes a low flow channel and a sediment sluiceway. The low flow channel also serves to provide upstream fish passage and allows for safe downstream boater passage. Low flow channel design considerations include a hydraulic analysis to verify hydraulic conditions will accommodate target fish species and timing.

The size and configuration of the low flow channel has been refined from the channel presented in the 2012 Preliminary Engineering Report based on typical anticipated late summer/early fall river flows of approximately 75 cfs. As designed, the low flow channel will pass approximately 30 cfs at a minimum depth of six inches. This design will allow for sufficient head to provide 45 cfs in the supply canal, and provide enough water depth at the weir crest to allow upstream fish passage as further discussed in Section 3.5 below.

3.2.1 Sediment Sluiceway

The sluiceway design incorporates a sluice gate at the entrance to control sluiceway flows, and a concrete channel bottom with grouted rock side slopes to provide a hydraulically smooth passage channel to move sediment away from the canal intake and through the diversion structure.

Morrison-Maierle developed a RiverFLO-2D model for both the the existing and proposed diversion structure to provide the basis for the hydraulic analysis of the rock ramp diversion and the sediment sluiceway. Based on this analysis, the sediment sluice gate will have an anticipated head differential of approximately 0.8 feet at bank-full flows (1,375 cfs) and up to 1.0 feet at the 100-year event (6,616 cfs). It is at these flows that transport of bed load past the diversion structure will be the most critical.

The anticipated head differential at the sluice gate translates to a sluice gate flow velocity of between 4.5 ft/s and 5 ft/s with the gate fully open. Anticipated velocities upstream of the sluiceway range from approximately 0.8 ft/s to 4.3 ft/s. From this velocity analysis, a significant fraction of bed load will deposit upstream of the diversion structure, and much of the bed load that remains entrained in the water column at the sluiceway will be readily transported downstream.

For significant runoff events transporting large volumes of bed load, it is likely that the depositional area upstream of the diversion will extend to and through the sediment sluice gate. This is due to the low relative flow velocity upstream of the structure (0.8 ft/s to 4.3 ft/s), and the small size of the sluice gate as compared to the overall length of the diversion crest. Under these circumstances, sediment deposition will likely occur in the sluiceway downstream of the sluice gate.

However, as river flows begin to drop, the head differential between the upstream and downstream sides of the sluice gate will increase, leading to a higher velocity through the gate. At a river flow of 500 cfs, the head differential increases to about 2 feet, resulting in a sluice gate velocity of approximately 7 ft/s. This higher velocity will provide for improved sediment transport potential.

Based on this analysis, it is recommended that the sluice gate be opened leading up to and during large runoff events, and be kept open for as long as flow conditions allow following a runoff event. This will maximize the quantity of bedload that can be transported through the sluiceway on an annual basis. The gate should be closed before the water depth over the diversion crest at the fishway falls below 6 inches or irrigation deliveries are impacted.

3.2.2 Diversion Dam Hydraulics

The RiverFLO-2D model provided an in-depth analysis that enabled the following items to be verified: available head for irrigation deliveries at high and low river flows, finalize rock riprap sizing, assess the capabilities and limitations of the sediment sluiceway, predict upstream fish passage at varying flows, and verify the scour potential downstream of the rock ramp diversion.

The RiverFLO-2D models were computed for the following stream flows in the Musselshell River: 75-cfs, 500-cfs, 1,375-cfs (2-year), and 6,616-cfs (100-year). These models assume no stoplogs are present in the existing sluiceway, the sediment sluice gate is closed in the proposed model, and no water was being diverted into the supply canal. The resulting rating curve for both the the existing and proposed diversion dam is shown in Table 3-1. This table shows the upstream water surface rising for low flows, and a small 0.2-foot rise for the 100-year flood event. The rise in upstream water surface for the low flows is expected since the stop logs are not present in the existing model.

Table 3-1 Diversion Dam Rating Curve

Flow	Upstream WS Elevation	
	Existing	Post-Project
(cfs)	(Feet, NAVD 88)	(Feet, NAVD 88)
75	3960.1	3964.2
500	3964.3	3965.1
1,375	3965.6	3966.0
6,616	3967.7	3967.9

Additional information from the model such as model layout, cross section plots, flow profiles and velocity distributions are shown in Appendix A. The velocity distributions are shown as both a color shaded overlay on the aerial mapping, as well as velocity vectors for the entire model and zoomed in on the diversion dam. The incremental flow (q) in cubic feet per second per foot is also displayed on an exhibit for the post-project conditions 6,616-cfs flow rate. This was calculated by multiplying the flow depth by the velocity for each node in the two-dimensional flow model. These values are very helpful to determine the riprap size, and scour potential.

3.2.3 Downstream Scour Potential

The scour along the downstream side of the proposed rock ramp diversion is calculated to ensure the structure will be stable. The scour is calculated using the methods described in the USBR document entitled: "*Computing Degradation and Local Scour*." This document presents the following methods for calculating scour below a diversion structure:

- Veronese (1937)
- Schoklitsch (1932)
- Zimmerman and Maniak (1967)

The potential scour was computed using these methods for all four modeled flow events. Table 3-2 presents a summary of the maximum scour elevation and it's coresponding flow event for each method. The scour calculations for each method are presented in detail in Appendix A3.

Table 3-2 Scour Calculations Summary

Method	Max Scour Elevation (ft, NAVD 88)	Max Flow Event (cfs)
Veronese (1937)	3956.4	75 cfs
Schoklitsch (1932)	3957.2	75 cfs
Zimmerman and Maniak (1967)	3950.4	6,616 cfs (100-yr)

The Veronese and Schoklitsch methods produce similar results for all flow rates, with the maximum computed scour occurring for the 75-cfs flows. The Zimmerman and Maniak method produces a simiar scour depth at the 75-cfs flow, but a much lower scour depth for the 6,616-cfs event. At a scour elevation of 3,950.4-feet, this is lower than the existing scour downstream of the existing sluiceway. Furthermore, the resulting scour downstream of the rock ramp is likely to be less than that downstream of the exsiting structure. Since the Zimmerman and Maniak method is so much different than the other two methods, and doesn't seem to match the historical operations, it is thrown out from consideration in this design.

3.3 Riprap Design

The rock ramp and pool and weir fishway will be composed primarily of angular rock riprap. The use of large diameter rock will significantly increase the hydraulic roughness of the ramp as compared to the hydraulic roughness of the natural channel upstream and downstream of the project. This will minimize downstream stream bed scour, improve recreational safety by avoiding the development of a large in-stream hydraulic jump, and improve the opportunity for upstream fish passage.

As shown in the design plans, the riprap in the area around the sluiceway and fishway will be grouted due to the complex horizontal and vertical rock layout in these areas. This will help ensure long-term stability in the most critical areas of the in-stream portion of the project. To the extent practical, interstitial spaces between the rocks will be maintained to improve the passage potential and to provide cover from predators for the smaller native non-game fish species and macroinvertabrates. The over-steepened portion of the rock ramp along the right side of the diversion will remain un-grouted.

3.3.1 Riprap Sizing

Since the right side of the rock ramp is un-grounted, the calculated flow conditions on this portion of the rock ramp are used for the riprap sizing. The riprap is sized using the 100-year flow event of 6,616-cfs. At this flow event, approximately 4,387-cfs is flowing over the diversion, and the remainder is in the right floodplain. The five methods used to calculate the median (D_{50}) riprap size are listed below:

- Abt and Johnson (1991)
- Robinson (1998)
- Ferro (1999)
- USACE (1991) Bed
- Whittaker and Jaggi (1986)

Each of these methods are presented and described in more detail in the United States Bureau of Reclamation's (USBR), "*Rock Ramp Design Guidelines*." An incremental flow rate of 40-cfs/ft was used for the riprap sizing, based on the maximum value found on the right portion of the rock ramp (Appendix A1). The design calculations for each of these methods are shown in Appendix A3, and are summarized in Table 3-3.

Table 3-3 Riprap Sizing Summary

Method	D_{50} (in)	D_{50} (ft)
Abt and Johnson (1991)	31.1	2.6
Robinson (1998)	23.8	2.0
Ferro (1999)	30.3	2.5
USACE (1991) Bed	64.9	5.4
Whittaker and Jaggi (1986)	24.2	2.0
Maximum =	64.9	5.4
Minimum =	23.8	2.0
Average =	34.9	2.9
Average (minus high and low) =	28.5	2.4
Design =	**30**	**2.5**

Based on this analysis, a median riprap size of 2.5 feet is used for this design. Using the recommendations in the USBR guidelines results in the riprap gradation shown in Table 3-4.

Table 3-4 Riprap Design Gradation

Percent Passing	Size	
	(in)	(ft)
100%	60	5
50%	30	2.5
20%	15	1.25

3.3.2 Riprap Source

Based on the anticipated riprap needs for this project, multiple local and regional rock sources have been investigated. Based on discussions with local contractors, public agency personnel, and DBWUA personnel, local sources of rock are limited, increasing the probability that rock will need to be hauled a significant distance (over 50 miles). From these investigations, three primary regional rock sources have been identified as having the anticipated size and quantity of rock needed for the project. Samples from each of these sources have been gathered and tested for suitability by Terracon. The parameters tested include specific gravity, absorption, and abrasion resistance. The results are presented in Table 3-5.

Table 3-5 Riprap Source Properties

	Riprap Source		
Parameter	**Harlowton Pit**	**Stenberg Pit**	**Casino Creek Pit**
Apparent Specific Gravity	2.64	2.60	2.56
Absorption	2.2	5.1	1.2
Los Angeles Abrasioin (% Loss)	16.1	48.5	16.7
Location	Harlowton	Big Timber	Lewistown
Approx. Distance to Pit (miles)	10	55	70

Testing results and photos of the rock material available from each of these sources is provided in Appendix C.

The project is located in an area where the geology is dominated by sedimentary rock deposits, primarily sandstone. Long-term in-stream performance of sandstones can be highly variable, and should only be used when other alternatives are not readily available or further testing is done to ensure that the binder material cementing the sandstone particles together is resistant to continuous immersion.

Both the Stenberg and Harlowton sources are sandstone. In addition, the abrasion loss of the rock from the Stenberg source is in excess of 40%, exceeding the upper maximum threshold as recommended by the US Army Corps of Engineers HEC-11 Design of Riprap Revetment circular.

The Harlowton 'pit' is a a non-developed rock outcrop adjacent to the hospital in Harlowton on Wheatland County property. Wheatland County has been making limited use of the readily available rock for various local projects. This site is the closest in proximity to the project site, and the abrasion resistance of the rock is quite good. Prior to selection as a source, we highly recommend additional testing be done to verify that the rock will function long-term with respect to the the behavior of the sandstone binder material under continuous immersion.

Also, significant development would have to occur at the Harlowton site to provide for the quantity of rock needed for the project. Due to the proximity of the site to the local hospital and the anticipated noise and dust related to source development and rock transportation, we recommend that further additional public outreach be underaken before considering this a viable source. This should include meetings with Wheatland County, City of Harlowton, and hospital personnel.

The third source tested is non-sedimentary rock from the Lewistown area available from Casino Creek Concrete. The rock is relatively hard, and has a very low absorption rate. Also, there is limited risk of material degradation due to long-term immersion in water. The one-way haul distance from this source is the longest of the three sites, but is is our recommendation that the Casino Creek rock be used for the project as it will provide the highest assurance of long-term project success.

The Casino Creek source contains adequate quantities of rock, can be obtained in the anticipated size range needed for the project, and does not entail additional source development.

Should DBWUA or another entity associated with the project become aware of a currently unknown developed source of suitable rock in the project vicinity and provide for testing, or a contractor bidding on the project provide documentation as to an equal or better substitute, any potential reasonable source should be considered for use at the site if it will improve project economics and meet meet the long-term requirements for the rock material.

3.4 Sediment Transport

It is desirable to avoid or decrease the intake of river sediment or bedload into the canal during high flow conditions. The sediment sluiceway will be a valuable component to help limit the amount of sediment transported to the Canal.

As proposed in this design, the sediment sluiceway includes a 4-foot by 4-foot slide gate with an inlet elevation 4-feet below the intake sill elevation of the headgate structure. Operator access to the control gate will be via an elevated structural walkway at an elevation of 3,968-feet, placing it approximately 1-foot above the calculated 100-year water surface elevation. This will allow gate operation at high river flows when use of the sluiceway will be the most critical to provide for sediment passage. Due to relatively higher approach velocities into the sluiceway gate with respect to adjacent cross-current intake velocities into the irrigation canal at high river flows, we anticipate that the majority of the sediment load in the area near the canal gates will be passed through the sluiceway and continue downriver.

As river flows begin to drop following spring runoff, sediment transport decreases and sediment passage becomes less critical. At the same time, irrigation demand begins to require a higher percentage of available flow. At that time, the sluiceway gate can be closed to provide for consistent irrigation deliveries through the irrigation season. This design provides an improvement over the current check board controlled notch in the diversion structure with respect to both sediment management and operator safety as the sluiceway gate will be accessible and operational at very high river flows.

Once sediment passes through the sluiceway gate, it will be conveyed downstream adjacent to the rock ramp structure utilizing a 30-foot long concrete-bottomed channel, that spills into a grouted riprap channel. The riprap portion of the sediment sluiceway is likely to be covered in sediment under normal operations. This sediment will then be cleaned from the structure during high events.

3.5 Fish Passage Criteria

Fish passage through this structure is an important benefit of the rock ramp design. The existing concrete weir wall is a major fish barrier that disrupts the river's natural ecologic function.

Fish passage goals for this project includes providing at least part-season passage for both non-native sportfish and native non-game fish. Based on input by Mike Ruggles of Montana Fish Wildlife & Parks at the project kick-off meeting on February 26, 2014, important species to be considered as a part of the fish passage design include brown trout (non-native sportfish), dace, and white sucker (native non-game fish). This list could also possibly be expanded to include ling, which though not currently present in the Musselshell River, may be reintroduced to downstream locations in the future.

Table 3-6 Fish Swimming/Jumping Abilities

Species	Burst Swimming Speed cm/s (ft/s)	Jumping Ability cm (ft)	Data Source (See Notes Below Table)	Spawning Time Frame
Brown Trout	250 (8.2)	45	1	Fall
Dace	71-82 (2.3-2.7)	15	2	Spring/Summer
White Sucker	80 (2.6)	15	2	Spring
Ling	80 (2.6)	10	1,2	Winter

1. US Forest Service Aquatic Organism Passage FishXing Sofware (3.0.20)
2. Experience/unpublished data source

Each of these fish species has different abilities relative to swimming speeds and jump heights which must be taken into consideration as a part of the passage design process. These fish species also have differing migration time periods for moving upstream to spawn. As a part of the 90% design, we have collected biological data for each of these species from published sources, unpublished academic sources, and previous project experience. Reliable data related to fish swimming and jumping abilities is severely limited in literature sources, especially with respect to non-game species. For the purposes of this 90% design effort, we are using the design data shown in Table 3-6 for each of the target species.

Data sets for the swimming and jumping abilities for non-anadromous fish species tend to present a wide range of variability. The values presented are conservative abilities for each of the species based on currently available information. As is demonstrated by Table 3-6, the fish species of interest typically move upstream to spawn at widely different times of the year, and have significantly variable abilities to negotiate a fish passageway.

Based on previous experience, hydraulic calculations, the relative abilities of the various fish species, and the seasonal variation in flows where passage is desirable, we believe that a pool and rock weir fishway will provide the best opportunity for fish passage within practical site constraints and project budget. As currently designed, the low flow channel/fishway average an approximately 8-inch drop between pools. The rock at each pool drop will be grouted in place to create a very short riffle/passage section allowing the smaller fish species with lesser jumping abilities to burst through localized channels between the rocks.

The pool and weir fishway design is not anticipated to provide full year passage for all targeted species at all flows. The intent is to provide appropriate design components in the low flow channel/fishway to allow passage windows for a particular species at the appropriate time of the year.

Results from the RiverFLO-2D modeling effort verify that the fishway design provides significantly lower velocities at all flows between 75 cfs (low flow) and 6,616 cfs (100-year event) than in-stream velocities across the diversion to the left and right of the fishway. Table 3-7 presents an overview of anticipated fishway velocities at various modeled flow rates, timing of the anticipated flow rates compared to each fish species preferred passage window, and the potential for fish passage during the anticipated passage window. This data is derived from the RiverFLO 2-D results provided in Appendix A1.

Table 3-7 Anticipated Fish Passage

Modeled Flow Event	Flow Rate (cfs)	Anticipated Timing of Flow Event	Species of Concern	Fishway Velocity (ft/s)	Fish Passage Potential
Low Flow	75	Fall/Winter	Brown Trout Ling	<2	Yes
	500	Pre-/Post Spring Runoff	Dace White Sucker	<3	Likely (See Discussion)
2 - Year	1,375	Spring	Dace White Sucker	~4	Marginal (See Discussion)
100 - Year	6,616	Spring	Dace White Sucker	~6	Unlikely (See Discussion)

As can be seen from review of the table, fall and winter passage of brown trout is achieved by the proposed design. Although not currently present in the system, it is likely that adult ling could also benefit from the proposed fishway should they be reintroduced.

The passage window for dace and white sucker is anticipated to occur before and after the spring runoff peak. At river flows of up to 500 cfs, the average velocity in the fishway is less than 3 ft/s. The fishway design incorporates gaps between the rocks at the weirs, and in all instances, a minimum of 6 inches of reveal will be maintained between top of grout and top of rock within the fishway. This design will create localized low velocity regions through each weir and along the bottom of the pools to allow for resting and cover areas to facilitate upstream passage of these species.

The RiverFLO 2-D model treats each rock weir as a smooth and level crest, so the velocities calculated by the model are conservative. The model calculations also do not reflect the presence of the interstitial gaps between the rocks. Based on these inherent limitations in the model and prior experience with this type of design, the probability of successful upstream passage of dace and white sucker at flows up to 500 cfs is high.

For flows nearer to bank full (2-year event or 1,375 cfs), velocities in the fishway begin to approach 4 ft/s. At this velocity, it will be more difficult for dace and white sucker to move upstream through the fishway. Some passage is still likely due to the design of the fishway providing for localized low velocity areas as discussed above.

Once flows begin to exceed the bank full event, the potential for successful upstream fish passage through the fishway for the species of concern likely present coincidental to high flows is limited. However, the river begins to access the right bank flood plain at flows exceeding the bank full event. Once river flows extend into the floodplain, any upstream passage will likely take place outside of the river channel and in the floodplain.

3.6 Structural Design

The structural design for the rock ramp diversion ensures that the concrete crest is stable and adequately connected to the existing concrete crest and slab under the anticipated loading conditions. The structural design of the diversion also includes the concrete wall and walkway for the sediment sluiceway. Structural design analysis has been completed per the standards set forth in the 2009 International Building Code, ASCE 7-05 Minimum Design Loads for Buildings and Other Structures, ACI 318-08 Building Code Requirements for Concrete Structures, and American Welding Society D1.1-04 "Structural Welding Code" as appropriate.

4 SAFETY CONSIDERATIONS

The proposed rock ramp design presents a significant safety improvement over the existing condition from both an operational and recreational standpoint. The updated headgates and associated structure will provide improved ease of access, operation, and handrails for operator safety. The sluiceway gate in the diversion structure provides a far safer operational alternative than the existing notch/check boards in the concrete structure. DBWUA personnel will be able to access and operate the sluiceway gate without having to enter the river.

Recreational safety will also be improved as the downstream hydraulics due to the rock ramp diversion will be considerably safer for boaters in the water than the existing condition. Smoother hydraulics will reduce the opportunity for boaters, swimmers, or operators to get trapped in a rolling standing wave below the diversion.

The most significant remaining public safety is related to the canal headgates and the sluiceway gate. As conceived, this project offers no protection for recreationists or others in the water near the left bank and the entrance to these gates. Under certain high flow conditions with a significant amount of water being diverted, it would be possible for a small boat or a person to be pulled into the canal gates. Depending upon the gate opening and/or any debris in the headgate structure, this could create a dangerous condition. We recommend that the project include signage to warn the public of this condition.

Also, at lower flow conditions, should the sluiceway gate be open, a large fraction of the in-stream water flow will be pulled through that gate. This could also lead to a public safety issue should a recreationalist get caught in this gate.

These potential safety risks are reasonably low as the project is located on private property, and recreational use on the Musselshell is limited. However, although the risk is small, it is our recommendation that this situation be further addressed by signage as a part of this project.

Additional alternatives to improve public safety related to the headgates and sluiceway are limited, and likely include some form of steel debris structure isolating the gates from the main river channel. This type of structure would need to be designed to withstand potentially significant impacts from floating debris, and will undoubtedly increase operational maintenance requirements. Icing concerns related to winter irrigation deliveries would also need to be considered as a part of any debris structure. Based on previous discussions with DBWUA personnel, it is understood that the water users would like to avoid incorporation of a debris or trash rack structure into the project.

5 AUTOMATION STUDY

The intent of this study is to evaluate options for improving operator efficiency of the four 6-foot by 6-foot slide gates at the canal headworks. Manual operation of gates of this size is a time and energy consuming endeavor. Therefore, considerable interest has been expressed by DBWUA in evaluating options to allow for more efficient and easier operation of these gates.

As is often the case with multiple gate diversions, for much of the irrigation season, primary control of irrigation deliveries can be maintained through frequent adjustment of two of the four gates. For this reason, it is our recommendation that automation is applied to only two of the gates. If desired, all four gates could be automated, but the increased project costs and long-term maintenance likely do not justify the installation at this time.

Typical headgate automation includes a wide array of options and costs depending upon site constraints, the needs of the operator, available power, and funding. Automated headgates can be operated with direct control, via connection to a gauging station and electronic controller to maintain a constant diversion rate, or remotely through a cellular service provider or radio connection to a base station. This report primarily addresses the direct control option.

Direct control will consist of a standard slide gate and threaded riser with the standard handwheel being replaced by a mechanical operator with reduction gearing to reduce power requirements. This type of geared operator can be driven by any one of the following options.

- Gasoline Powered Portable Actuator
- Electrically Powered Portable Actuator
- Alternating Current Motor and Drive Assembly
- Direct Current Motor and Drive Assembly

Each of these options has advantages and disadvantages at this site as described below.

<u>Gasoline Powered Portable Actuator</u>

A gasoline powered portable actuator combines a geared drive mechanism with a small 2-cycle gasoline powered engine on a stand. This type of actuator is designed to be a portable unit that can be readily moved between individual gates. The operator would typically be transferred to and from the diversion location by the person responsible for making gate adjustments. A data sheet for a standard gasoline powered operator is provided in Appendix D.

A typical operator would weigh under 40 pounds, and produce sufficient torque to provide gate operation under typical operating conditions. Following is a list of advantages and disadvantages related to this type of actuator.

Advantages

1. Portable for off-site maintenance and storage.
2. Relatively cost effective.

Disadvantages

1. Water Manager will be required to haul operator to and from diversion site, and provide off-site storage for unit, or construction of an on-site storage facility will be required.
2. Operator will require ongoing maintenance through the irrigation season, and annual maintenance related to winter storage.
3. Operator will have continuous fuel costs, and will require Water Manager to maintain a supply of 2-cycle fuel.

Estimated Cost: $3,000

Electrically Powered Portable Actuator

An electrically powered portable actuator combines a geared drive mechanism with a small electric motor, typically operating on standard 120 Volt alternating current. Similar to the gasline powered actuator, this type of actuator is designed to be a portable unit that can be readily moved between individual gates. The operator would typically be transferred to and from the diversion location by the person responsible for making gate adjustments. This type of operator typically requires a portable gasoline generator to supply power.

Two different variants of an electrically powered actuator could employed. Specially designed actuators are available from headgate manufacturers to serve this purpose. These actuators are geared and powered appropriately for this purpose, and typically come with an adjustable height stand to hold the unit in place during gate operation. This type of actuator can weigh as much as 70 pounds, making it less portable than a gasoline powered operator.

The second possible variation is to utlize a high quality ½" drive variable speed corded drill with an appropriate adaptor for connection to the operating nut on the headgates. This variation will also require a gasoline powered generator for operation. As this use is not the original design purpose for a corded drill, we do not recommend this approach for gates of this size. A drill with sufficient power to operate one of the gates on the project can be a danger to the operator, or damage the lifting stem, if the gate jams and the drill does not have adequate torque limiting capability. Maximum input torque should be limited to about 75-foot pounds.

A data sheet for a standard electrically powered operator is provided in Appendix D. Following is a list of advantages and disadvantages related to this type of actuator.

Advantages

1. Portable for off-site maintenance and storage.
2. Relatively cost effective.

Disadvantages

1. Generator is required for operation.
2. Water Manager will be required to haul operator and generator to and from diversion site, and provide off-site storage for unit, or construction of an on-site storage facility will be required.
3. Electrically actuated operator should require less maintenance than a gasoline powered actuator. However, the time and cost to maintain the associated generator will offset any maintenance reductions related to only the actuator.
4. Safety concerns related to the use of a ½" drill as an actuator.

5. Generator will have continuous fuel costs.

Estimated Cost: $4,000 (Proprietary actuator plus generator)
$1,500 (1/2" drill with adaptor, plus generator)

Alternating Current Motor and Drive Assembly

This type of system would employ an electric motor and drive assembly mounted directly to each headgate or group of headgates. This type of system typically consists of a self contained unit provided by the headgate manufacturer. This option would require a ready source of commercial 120 Volt, 60 Hertz power or the use of a portable generator. There is currently no power source available at the site, and the cost of bringing in power would be high as the nearest existing power source is approximately one mile to the west.

This type of system is efficient to operate, and provides adequate power to open and close the gates under a wide range of operating conditions. Without a nearby accessible power source, a portable generator will be required to utilize this optioin.

Following is a list of advantages and disadvantages related to installing an alternating current motor and drive assembly for headgate operation.

Advantages

1. Pre-packaged unit designed specifically for the project by the gate manufacturer.
2. Provides basis for future additional automation, control, and telemetry upgrades.
3. Provides sufficient power to efficiently operate headgates under typical river flow conditions.
4. Most systems allow manual gate operation.

Disadvantages

1. Generator will be required for operation.
2. Potential for higher maintenance costs as compared to other options.

3. Relatively high capital cost.

Estimated Cost: $10,000 (Per Headgate with Generator)

Direct Current Motor amd Drive Assembly

This type of system would employ an electric motor and drive assembly mounted directly to each headgate or group of headgates, similar to the alternating current option discussed above. However, the motor would utilize 12 Volt direct current power. This power requirement would allow the drive motor(s) to be operated using a running vehicle engine with wired temporary leads to the vehicle battery, a 12 Volt deep cycle battery (short term), or a generator.

This type of system will typically operate much slower than any of the other alternatives due to the power limitations of a 12 Volt source. Operating speed can be expected to be in the 10 to 20 rpm range, versus several times that for the other options presented.

One benefit to this type of system is that it can be readily adapted in the future for use with solar power and automated headgate control via downstream canal flow measurement and/or off-site telemetry.

The cost for this type of system is highly variable. Pre-packaged “off the shelf” options are at the high end of the cost spectrum, while lower cost options are available that are field assembled from readily available components from multiple sources.

A photograph of an example installation is provided in Appendix D. The example shown includes additional hardware for automated gate control through flow measurement and telemetry, but the premise is similar.

Advantages

1. Permanently fixed to headgate structure eliminating the need for off-site transfer and storage.
2. Low cost, readily available, and multiple power source options.

3. Provides basis for future additional automation, control, and telemetry upgrades.
4. Non-proprietary components allowing for relatively straightforward and low cost maintenance and repair.

Disadvantages

1. Relatively lower operating speed than other options.
2. Motor(s) permanently installed at headgate location require weather protection for longevity.
3. Limited power output of 12 Volt DC motor may not be able to readily free up stuck or frozen headgates.
4. Depending upon installation, manual operation of gates could be difficult without disconnecting motor drive.

Estimated Cost: $2,500 - $10,000 (Per Headgate)

The low end of the price range is based on using off-the-shelf components including a 12 Volt DC motor, chain drive mechanism, and enclosure without a power supply. This is based on the assumption that a vehicle battery or other power source would be used to allow gate operation. Higher end pre-packaged systems are fully self contained, powered, and geared for operation of a specific gate size. These systems also easily accommodate manual gate operation.

The scope of this design effort is to provide the above gate automation options to present the DNRC and DBWUA with a general overview of readily available and potential technologies. Based on feedback from the DNRC and DBWUA regarding these options as a part of the 30% design review process, it is our understanding that the preferred option is to utitlize an altenating current motor and drive assembly.

It is our recommendation that any automation project include at least two of the headgates. A high level of water delivery control can typically be maintained with primary adjustment of half of any available gates. Motorized control of all four gates could be completed if desired, but the benefits brought about should be anticipated to be marginal with respect to the increased capital costs and long-term maintenance concerns.

To this end, headgate procurement specifications have been developed to include a line item for vendors to provide pricing for alternating current gate operators. Following the headgate bid process, the DBWUA can determine if the additional project cost is worth the increased capital investment.

6 PRELIMINARY COST ESTIMATE

We have developed a preliminary construction cost estimate based on the 90% design effort. Unit costs for the various project components are based on industry standards and previous experience. The cost estimate includes a 15% contingency to allow for changes in material costs and unlisted items in the cost calculations. The cost estimate will be refined and improved as a part of the final design.

90% Design Estimated Construction Cost: $757,942

The unit costs and calculations used to arrive at the estimated construction cost are provided in Appendix E.

7 CONCLUSIONS

The proposed 90% design as described in this report has incorporated the available project data with input provided by project stakeholders to develop a project that will meet the long-term operational needs of DBWUA. The design provides for the primary intended function of the project to deliver irrigation water, while providing for improvements related to ease of operation and public safety when compared to the existing condition.

As proposed, the new headgate layout will provide for significant water control and operational improvement. This includes addressing high water, debris, and icing concerns by providing for better equipment access to the headgates. The design also incorporates features that will improve sediment management at the diversion, and allow for seasonal upstream fish passage within practical budget constraints.

As a part of finalizing the design and moving toward the construction of this project, we request that action be taken or input be provided relative to the following items.

- Final confirmation that the overall concept of the proposed design meets the needs and expectations of DBWUA and DNRC.
- Provide suggestions for changes or improvements to the 90% design that will better meet the goals of DBWUA and DNRC.
- A decision be made with respect to moving forward with headgate procurement as soon as practical.
- A determination be made as to the means of riprap sourcing and procurement.

With a timely response to the above requested action items, Morrison-Maierle will remain on schedule to deliver project plans and specifications in time for an August 2014 construction start.

8 REFERENCES

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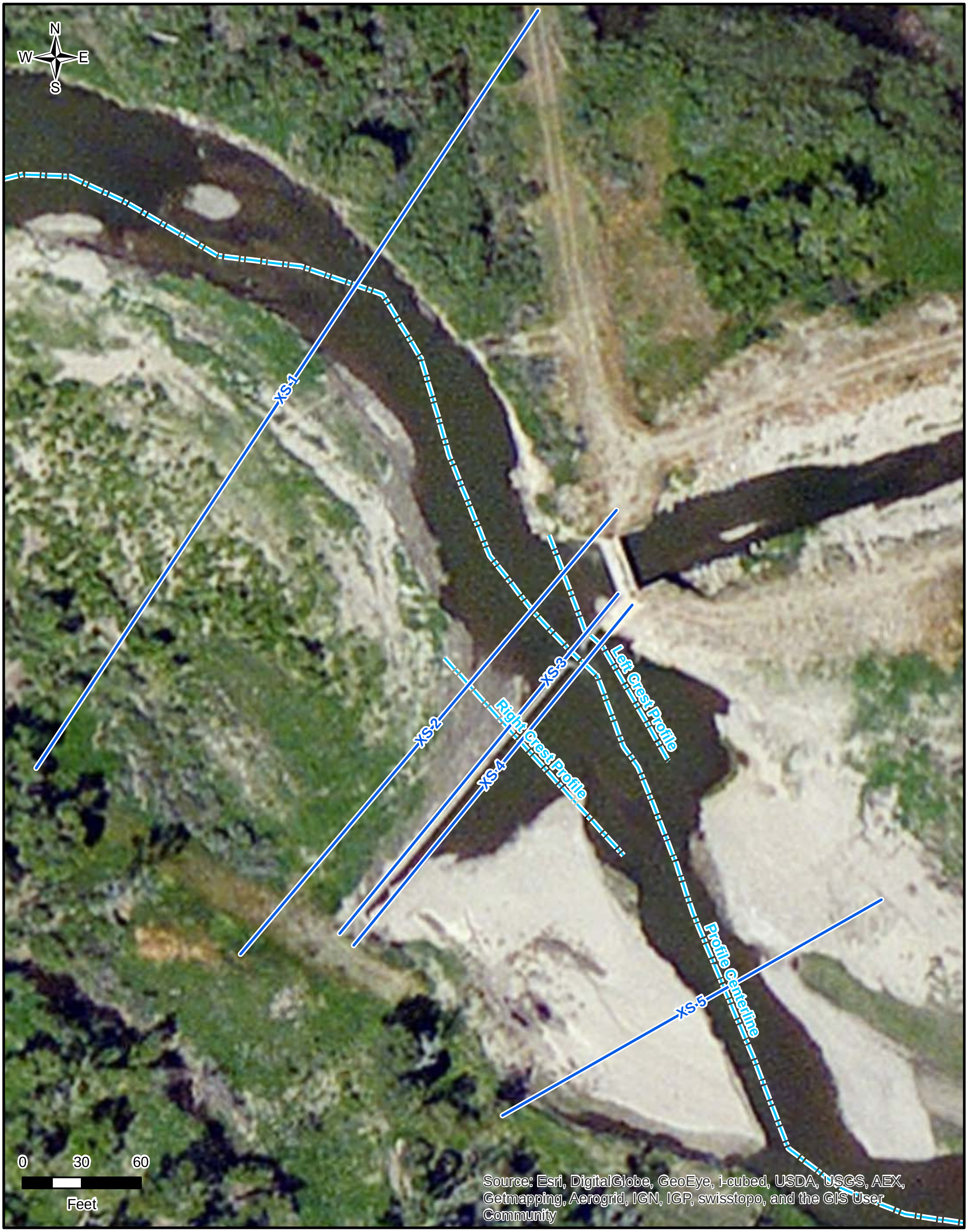
APPENDIX A1
DIVERSION DAM HYDRAULICS



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DRAWN BY: JTS
CHK'D BY: NK
APPR. BY: KS
DATE: 5/23/2014

DEADMAN'S BASIN DIVERSION DAM AND HEADGATE REPLACEMENT PROJECT

WHEATLAND COUNTY MONTANA

PROJECT NO. 1447.035

FIGURE NO. A

RiverFLO-2D Model Layout

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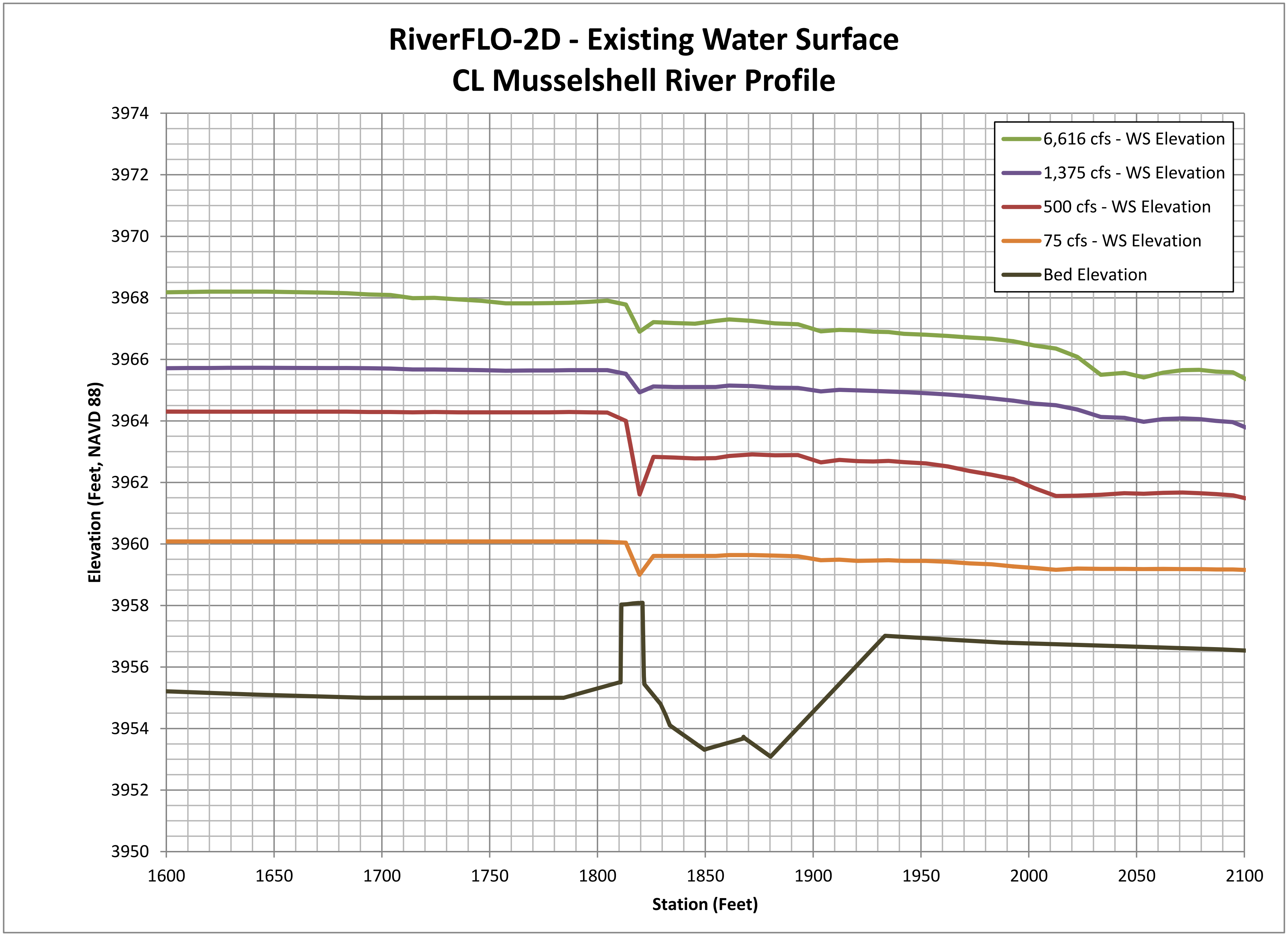

RiverFLO-2D - Existing Water Surface
CL Musselshell River Profile
6,616 cfs - WS Elevation
1,375 cfs - WS Elevation
500 cfs - WS Elevation
75 cfs - WS Elevation
Bed Elevation
Elevation (Feet, NAVD 88)
Station (Feet)
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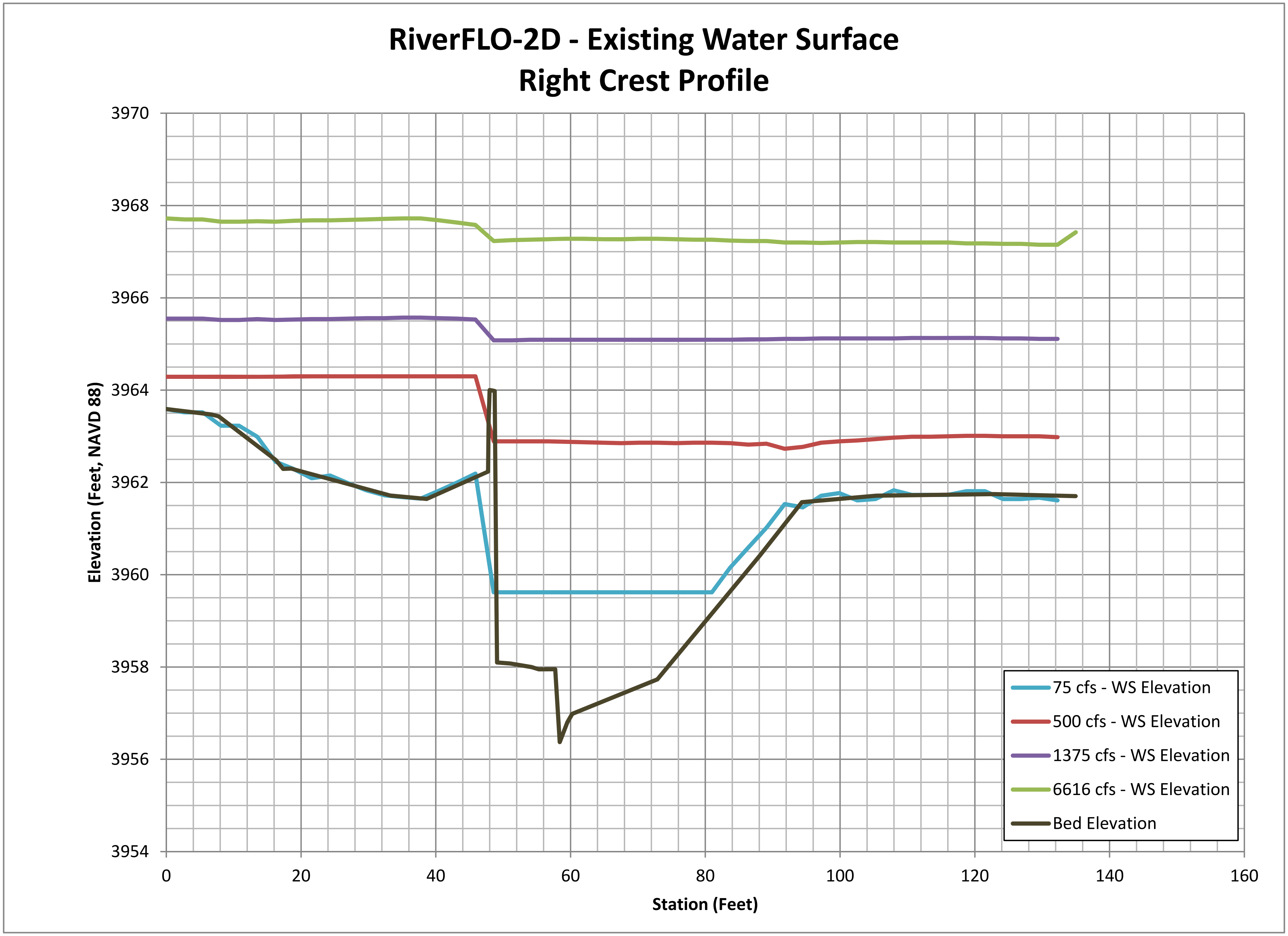
2100



RiverFLO-2D - Existing Water Surface
Right Crest Profile
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Elevation (Feet, NAVD 88)
0
20
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100
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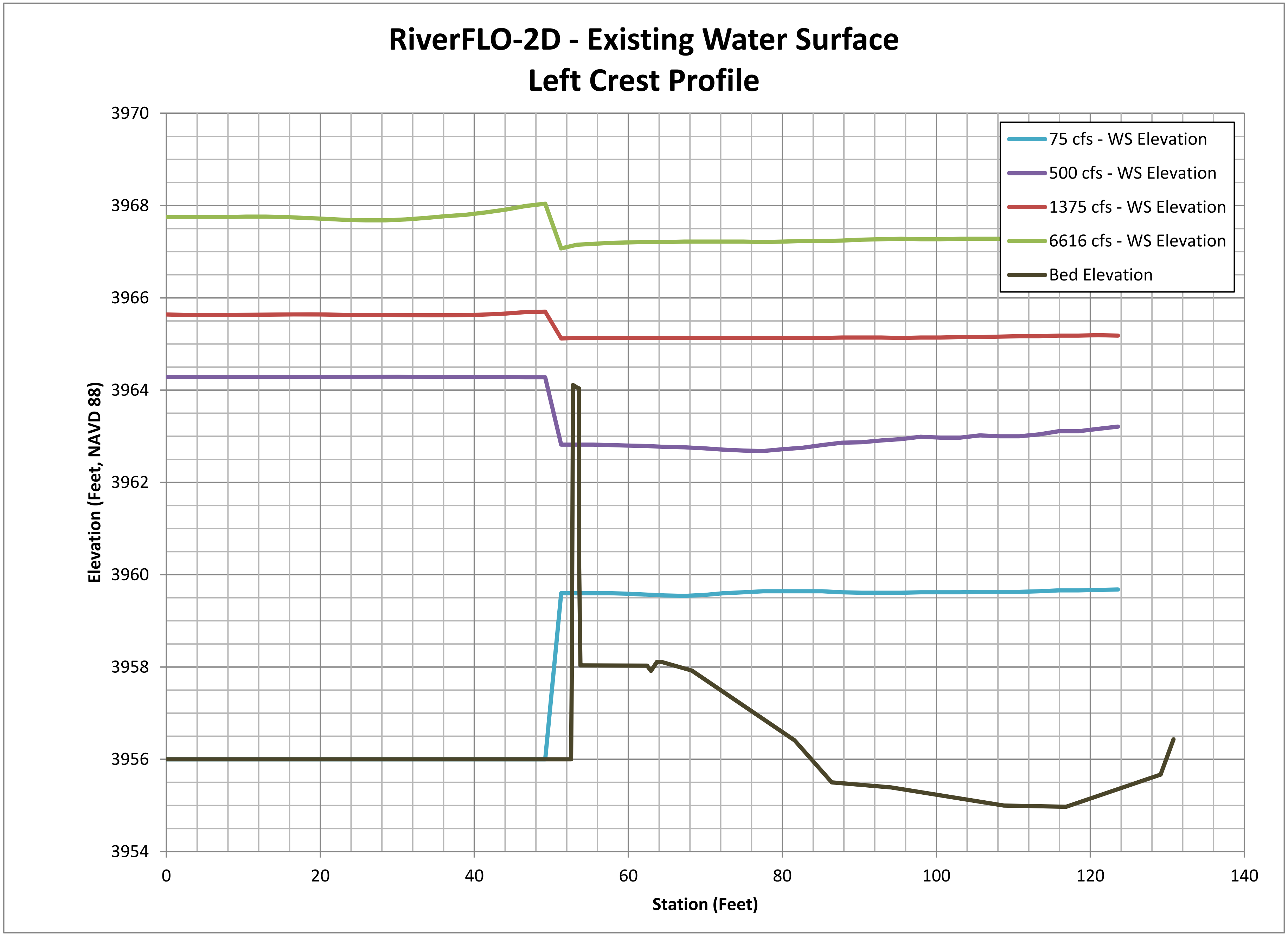
140
160
Station (Feet)
75 cfs - WS Elevation
500 cfs - WS Elevation
1375 cfs - WS Elevation
6616 cfs - WS Elevation
Bed Elevation


RiverFLO-2D - Existing Water Surface
Left Crest Profile
75 cfs - WS Elevation
500 cfs - WS Elevation
1375 cfs - WS Elevation
6616 cfs - WS Elevation
Bed Elevation
Elevation (Feet, NAVD 88)
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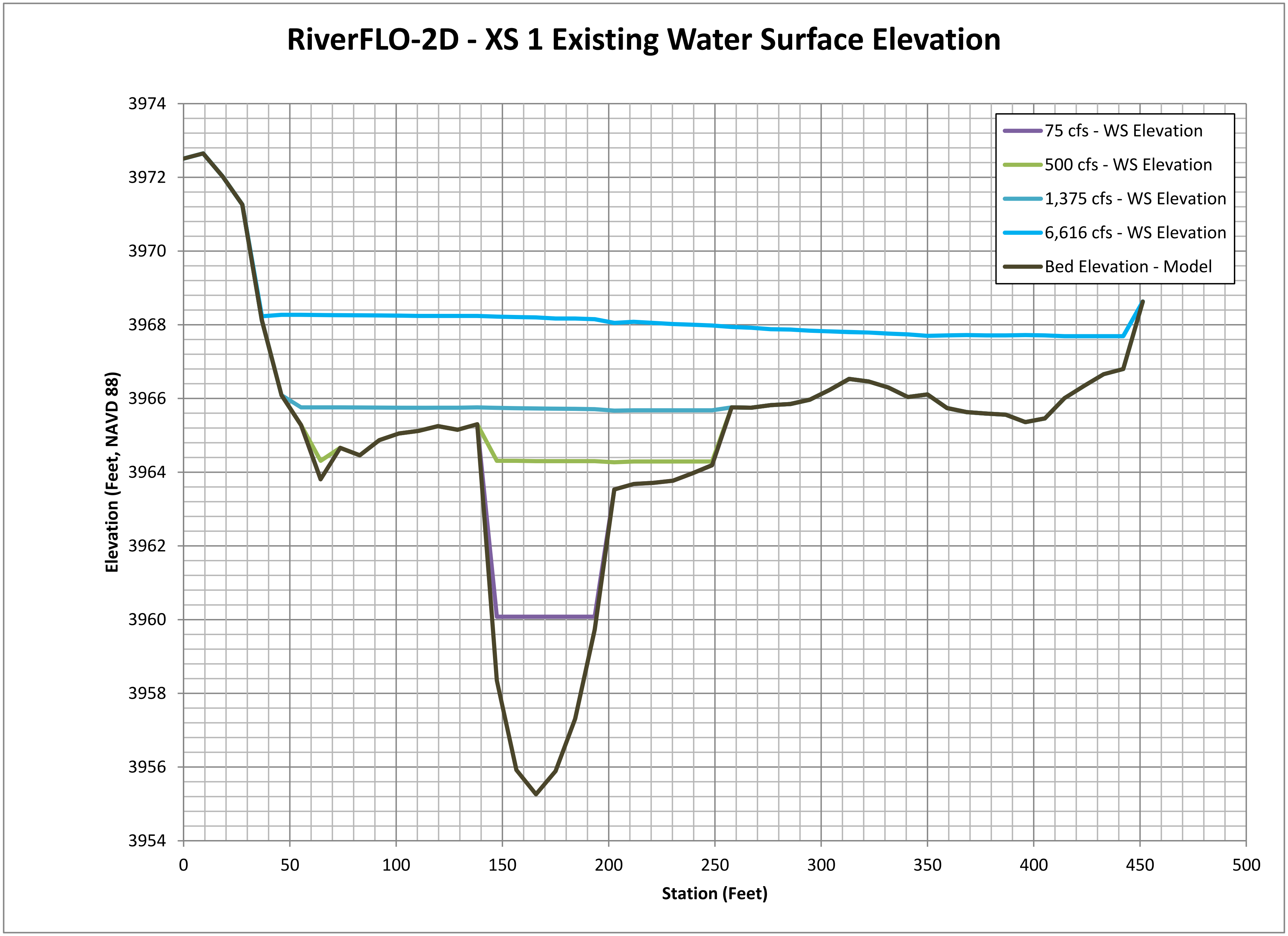
Station (Feet)


RiverFLO-2D - XS 1 Existing Water Surface Elevation
Elevation (Feet, NAVD 88)
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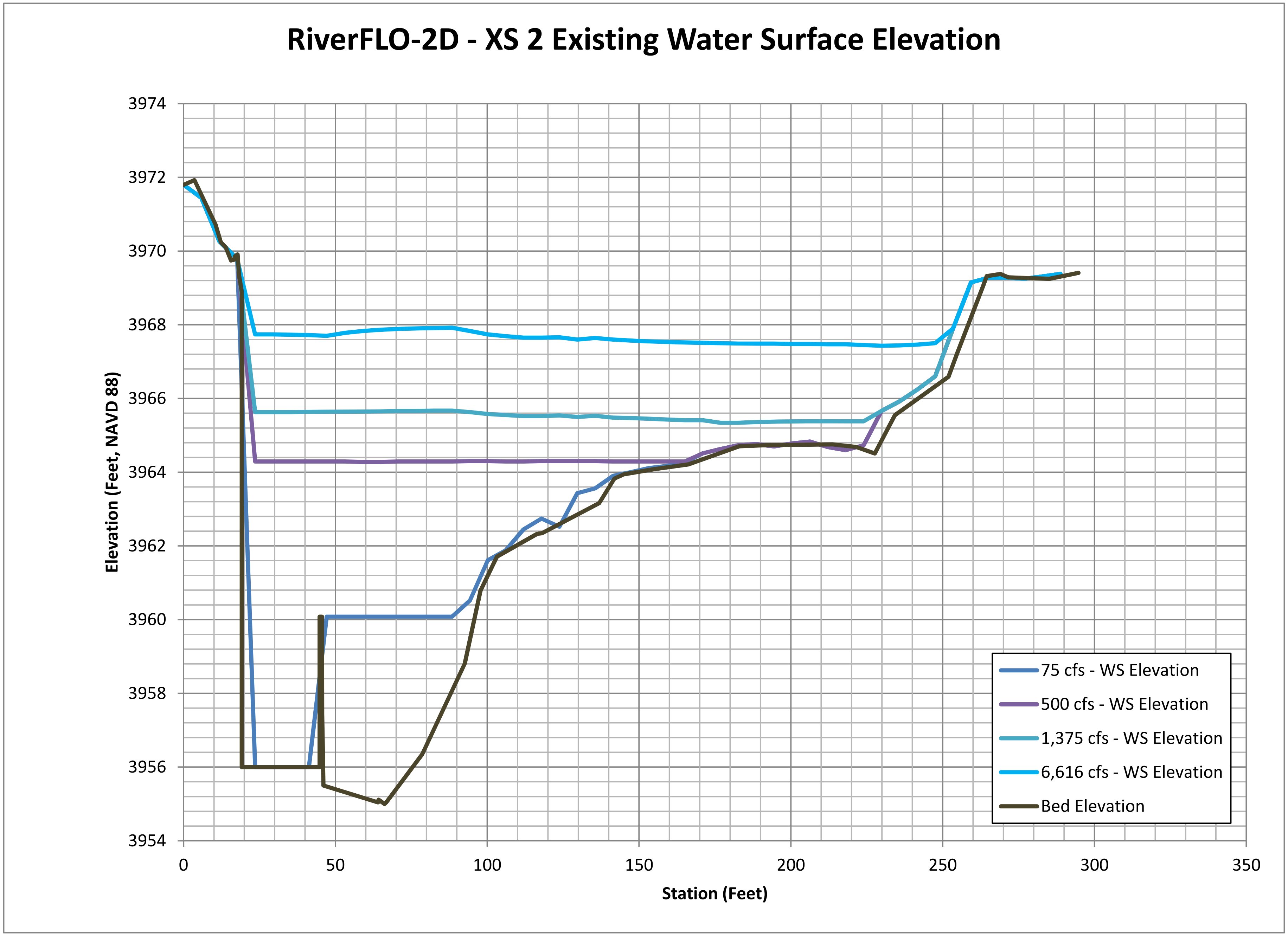
Station (Feet)
75 cfs - WS Elevation
500 cfs - WS Elevation
1,375 cfs - WS Elevation
6,616 cfs - WS Elevation
Bed Elevation - Model



RiverFLO-2D - XS 2 Existing Water Surface Elevation
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Elevation (Feet, NAVD 88)
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200
250
300
350
Station (Feet)
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500 cfs - WS Elevation
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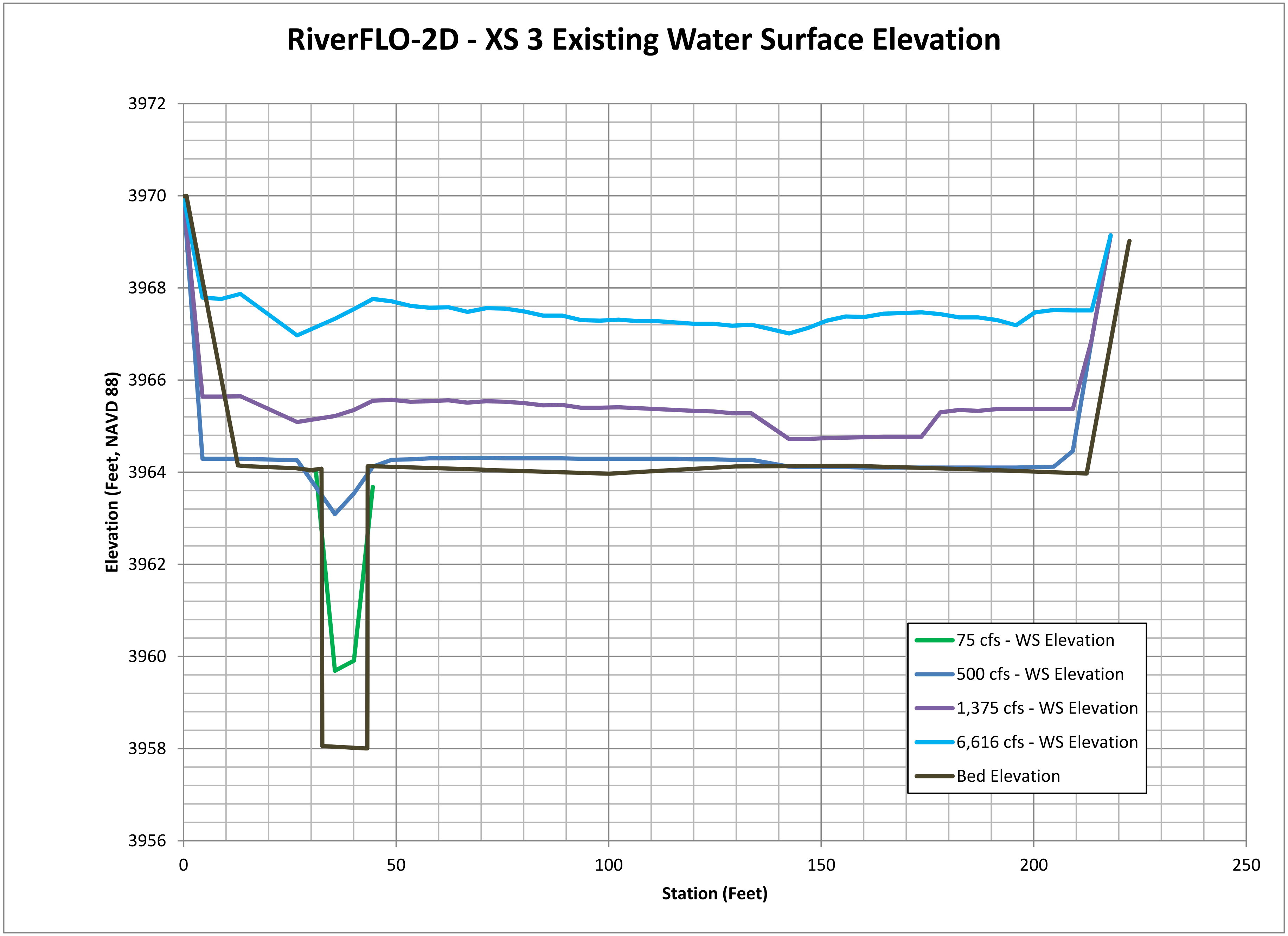
6,616 cfs - WS Elevation
Bed Elevation


RiverFLO-2D - XS 3 Existing Water Surface Elevation
Elevation (Feet, NAVD 88)
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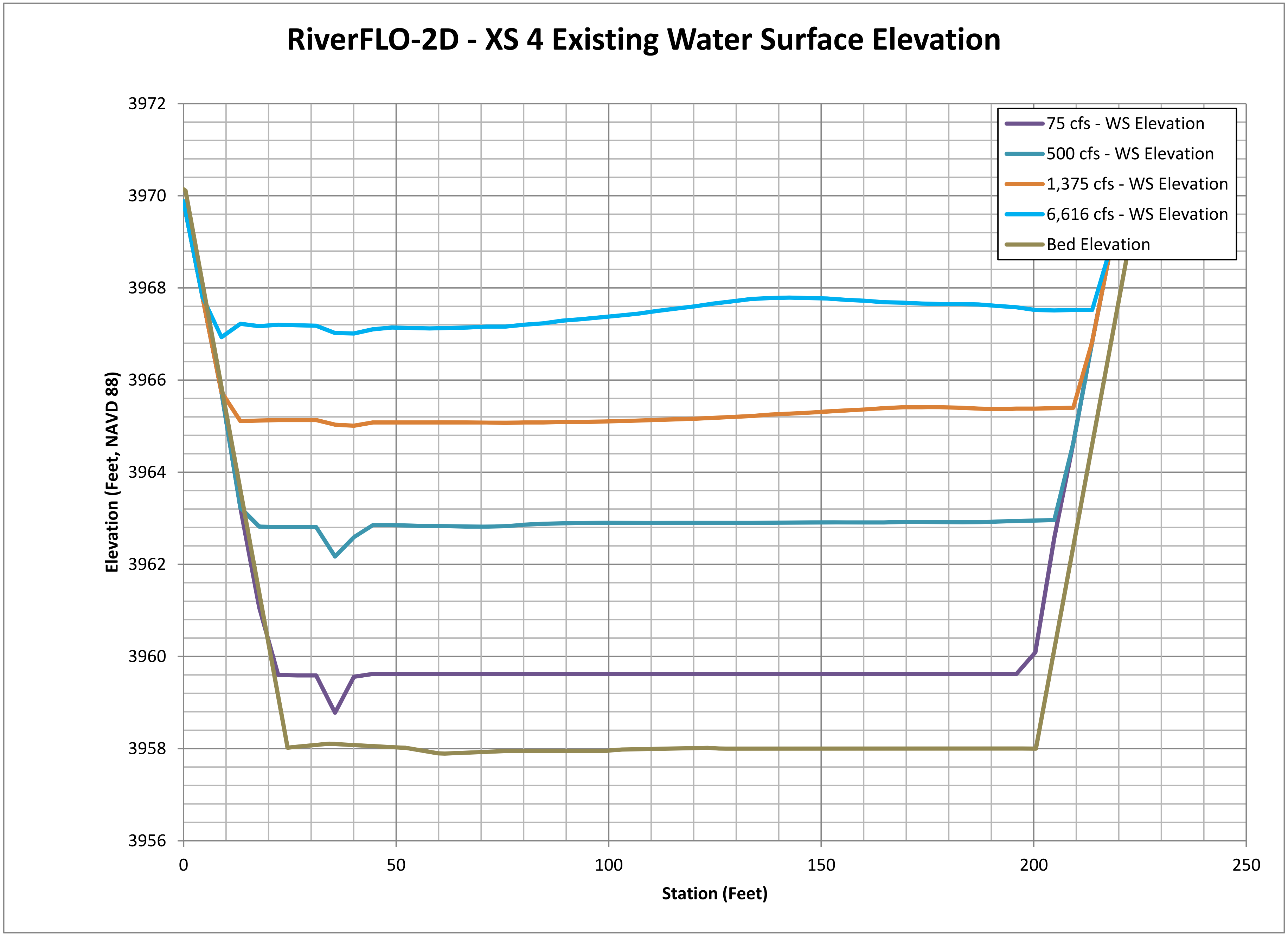
Station (Feet)
75 cfs - WS Elevation
500 cfs - WS Elevation
1,375 cfs - WS Elevation
6,616 cfs - WS Elevation
Bed Elevation



RiverFLO-2D - XS 4 Existing Water Surface Elevation
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500 cfs - WS Elevation
1,375 cfs - WS Elevation
6,616 cfs - WS Elevation
Bed Elevation
Elevation (Feet, NAVD 88)
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Station (Feet)
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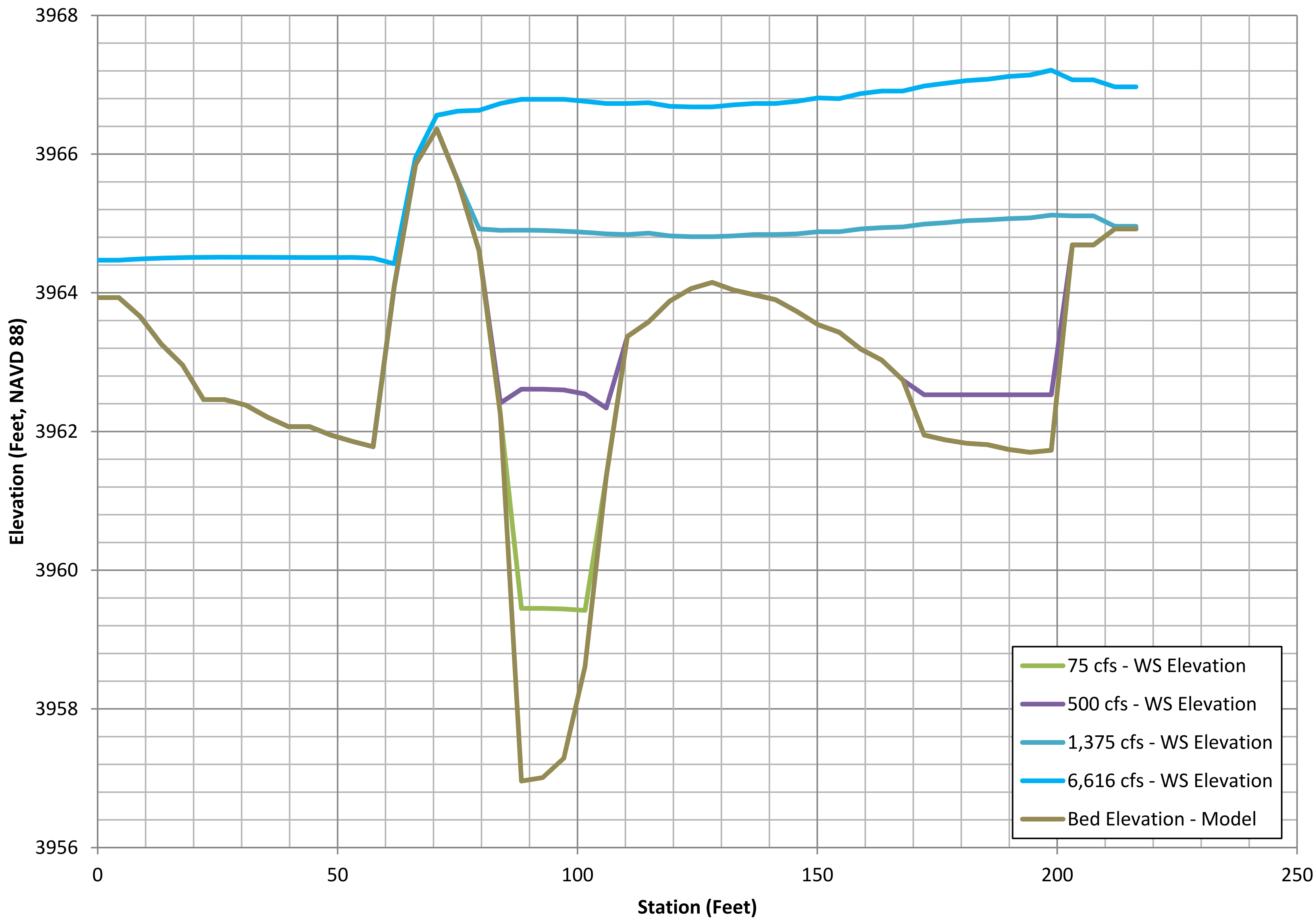
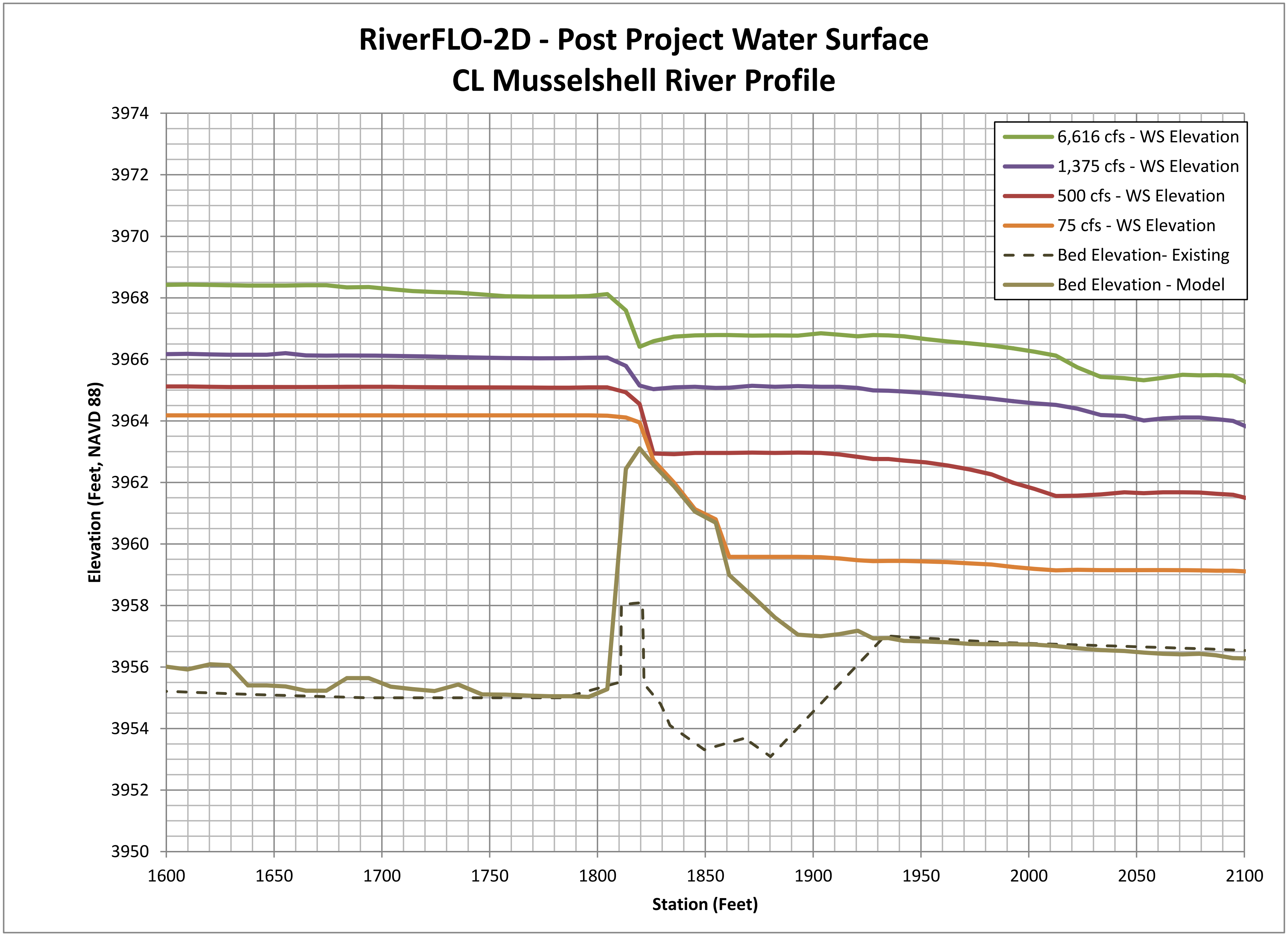
RiverFLO-2D - XS 5 Existing Water Surface Elevation




RiverFLO-2D - Post Project Water Surface
CL Musselshell River Profile
6,616 cfs - WS Elevation
1,375 cfs - WS Elevation
500 cfs - WS Elevation
75 cfs - WS Elevation
Bed Elevation- Existing
Bed Elevation - Model
Elevation (Feet, NAVD 88)
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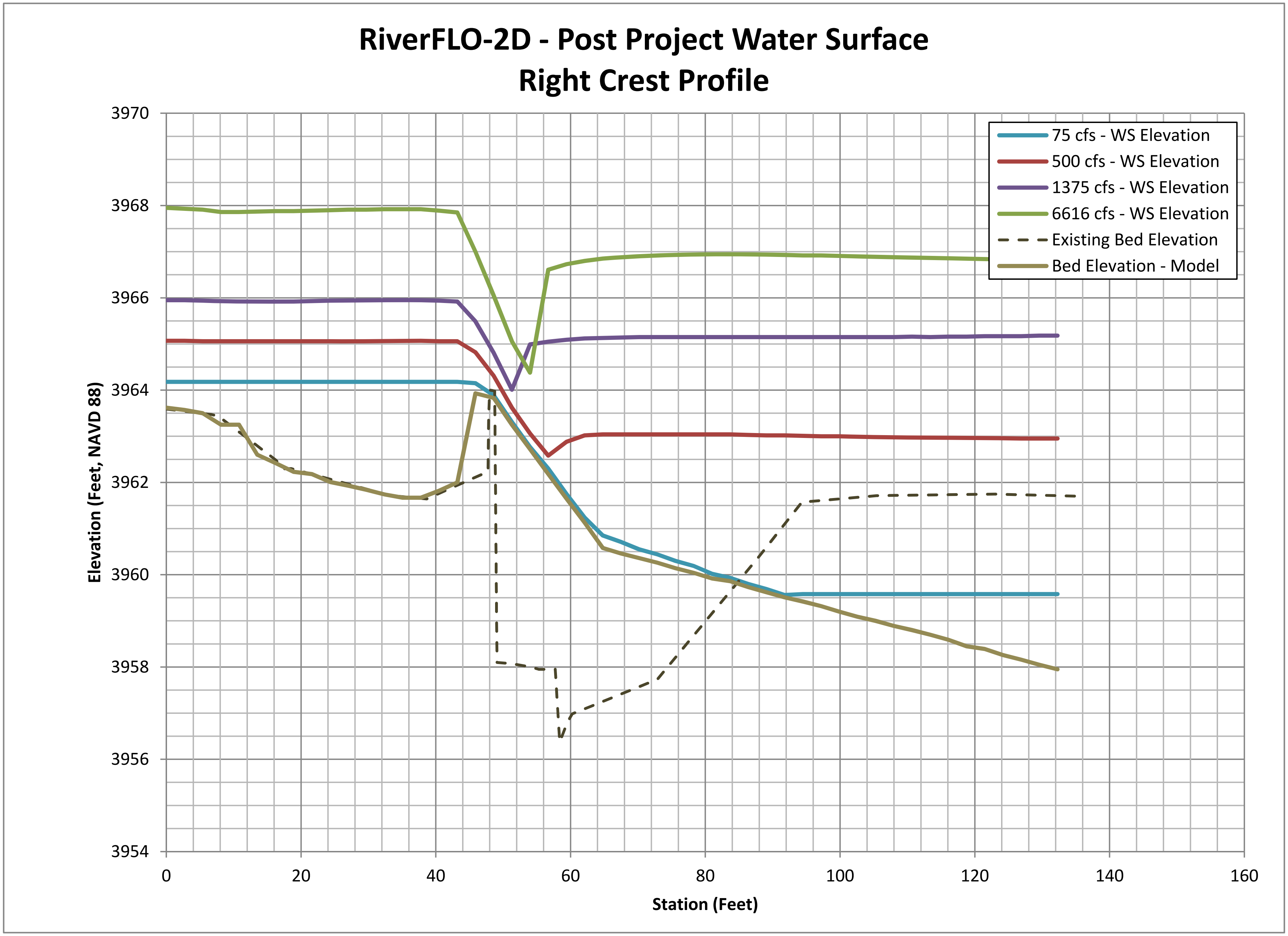

RiverFLO-2D - Post Project Water Surface
Right Crest Profile
75 cfs - WS Elevation
500 cfs - WS Elevation
1375 cfs - WS Elevation
6616 cfs - WS Elevation
Existing Bed Elevation
Bed Elevation - Model
Elevation (Feet, NAVD 88)
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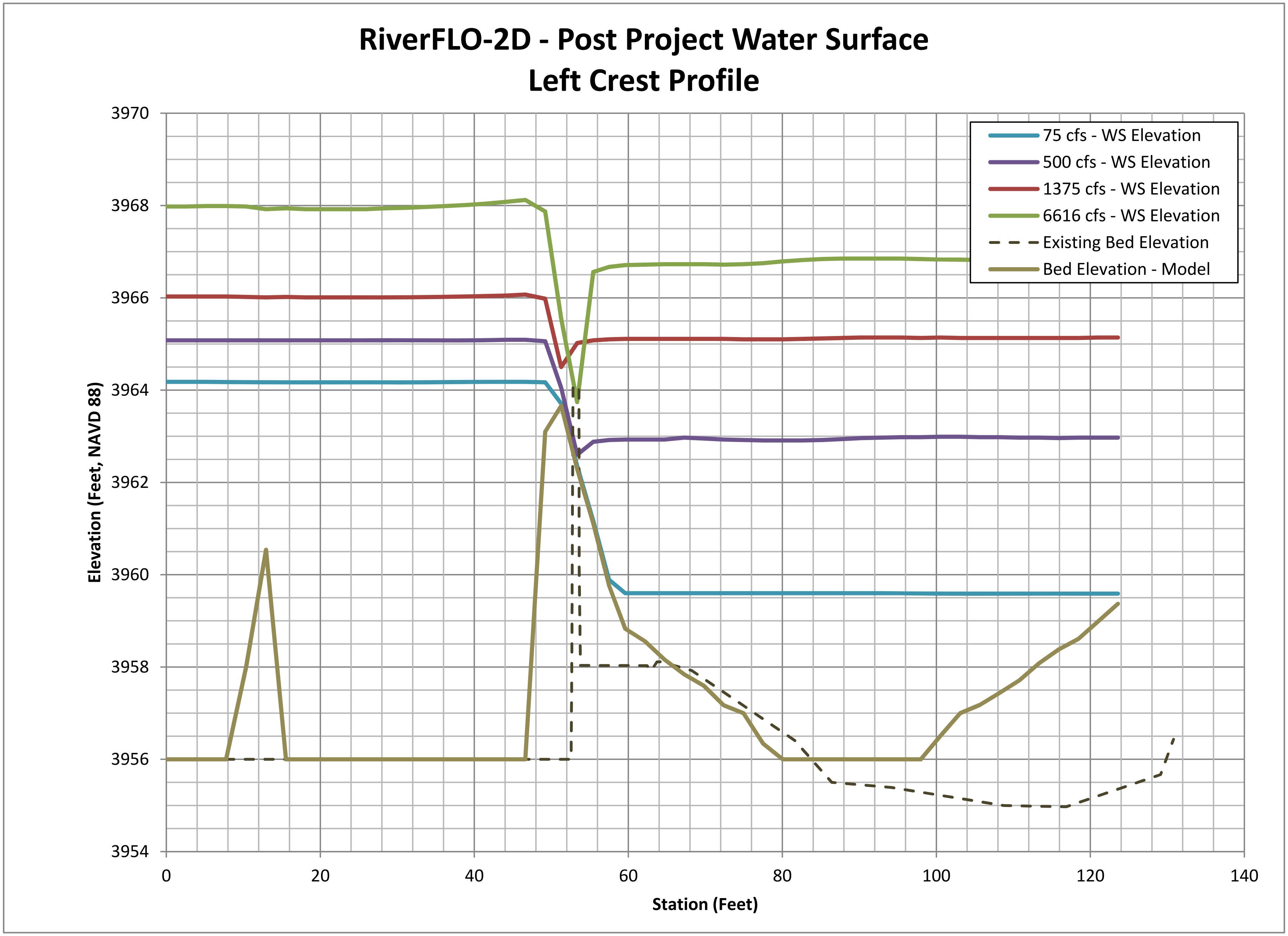
Station (Feet)


RiverFLO-2D - Post Project Water Surface
Left Crest Profile
Elevation (Feet, NAVD 88)
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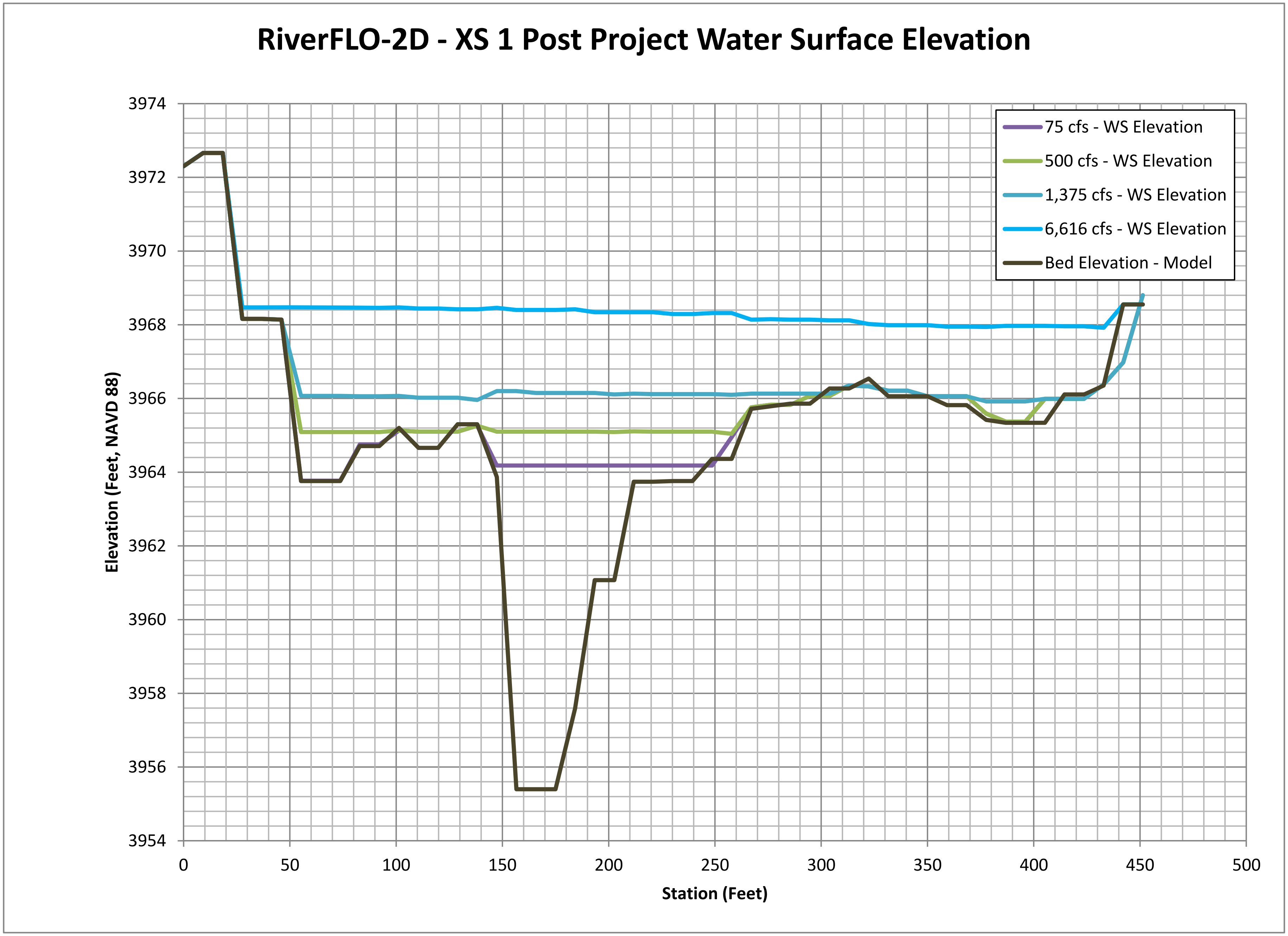
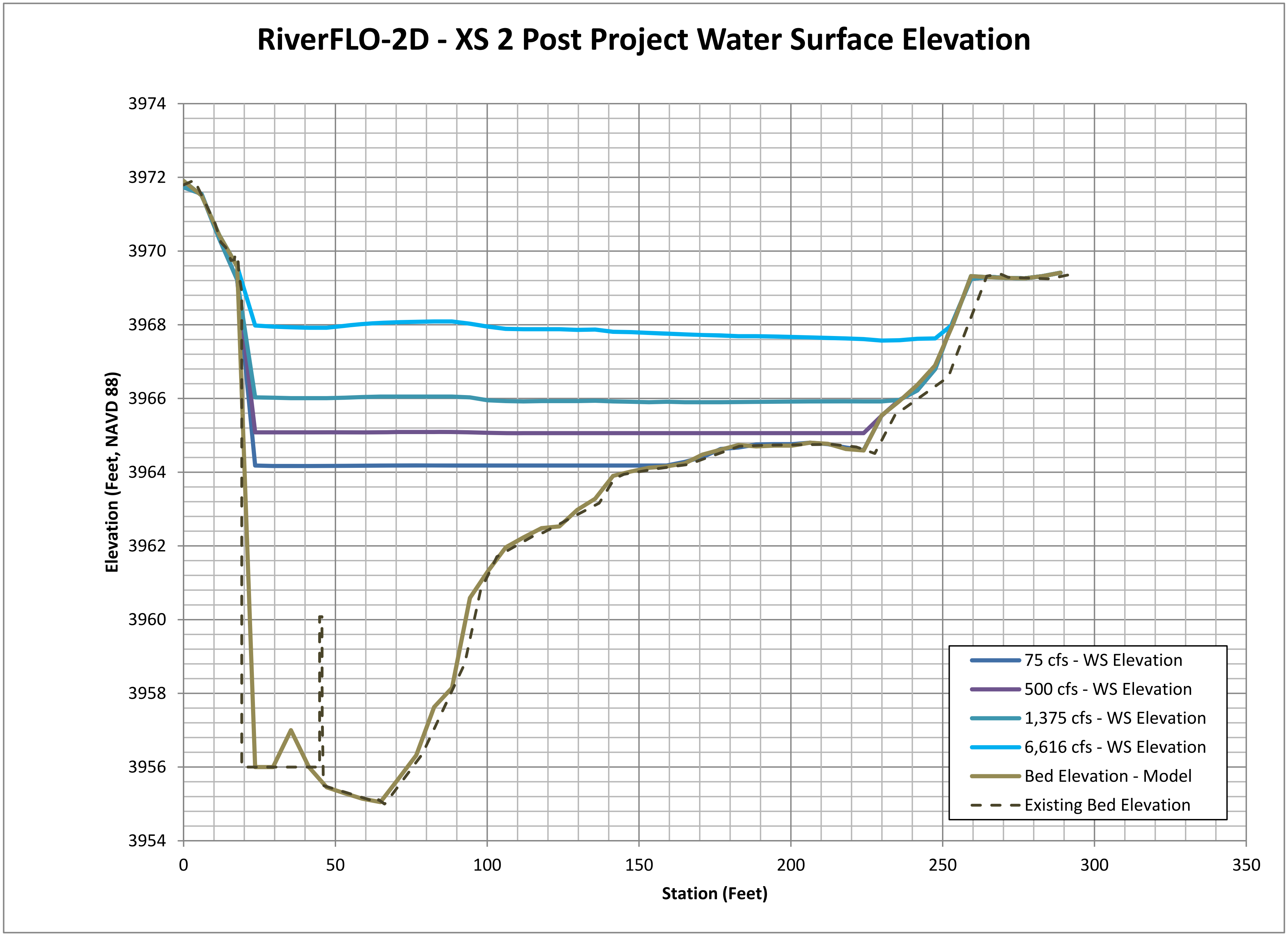
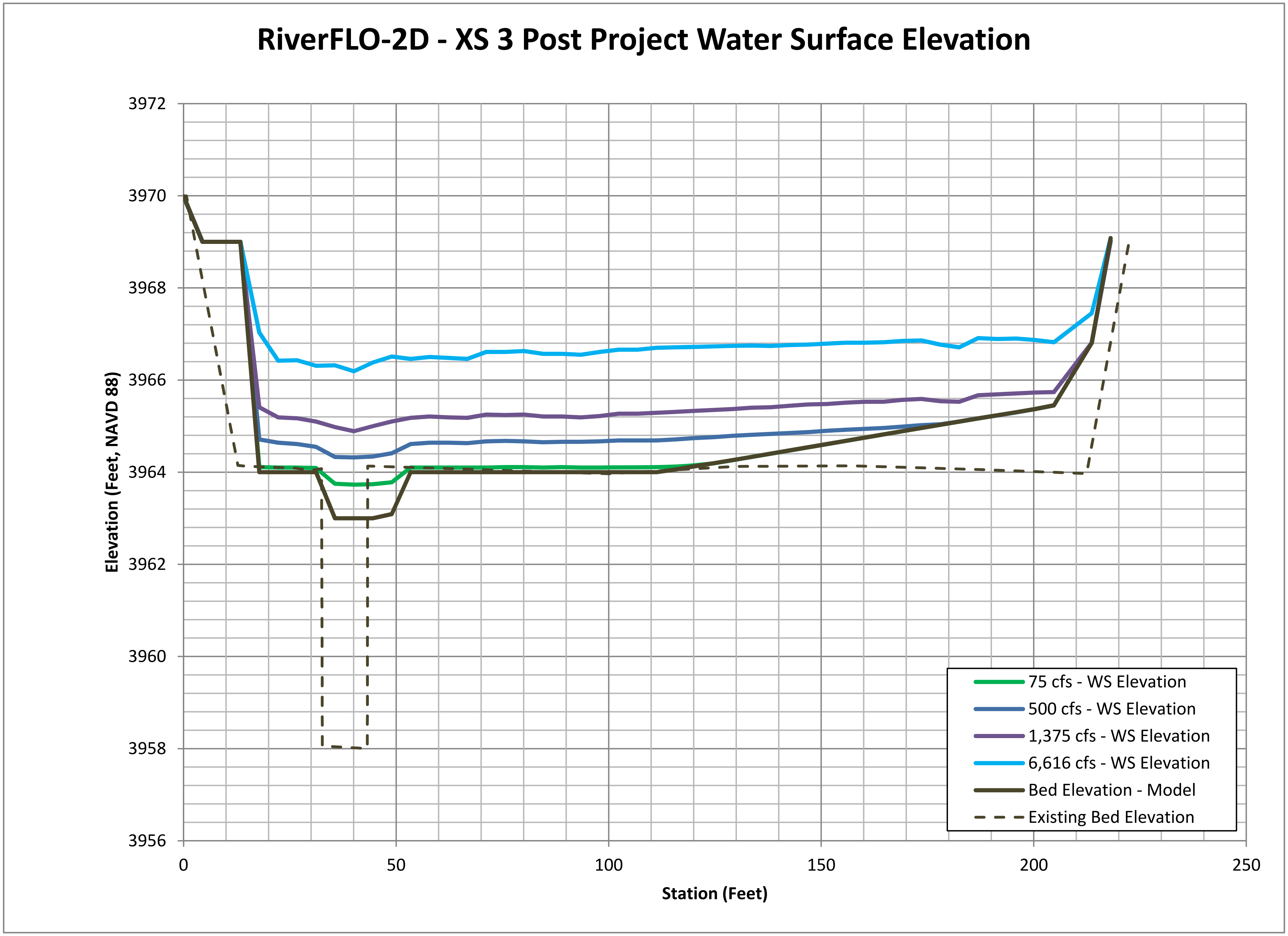
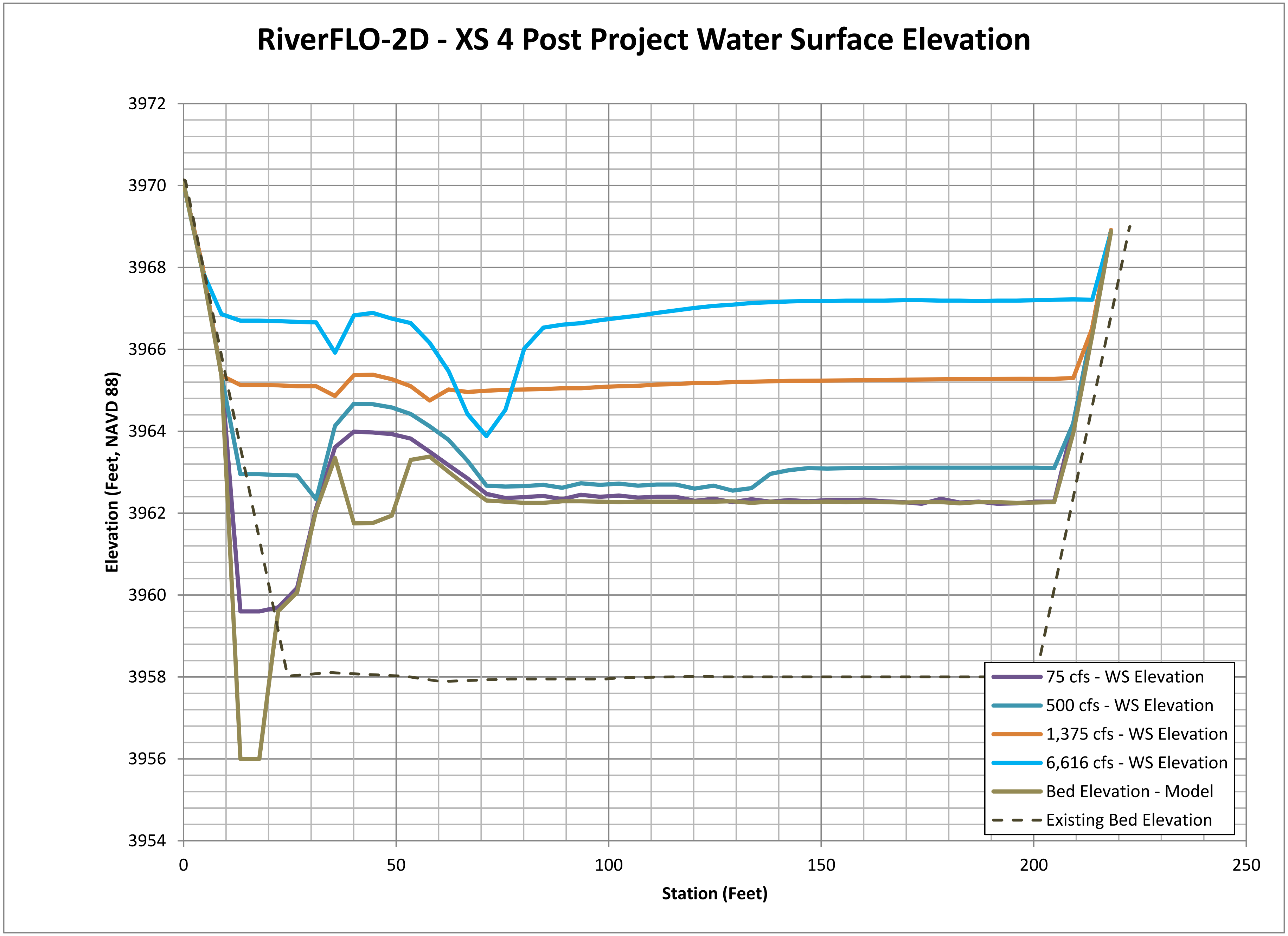
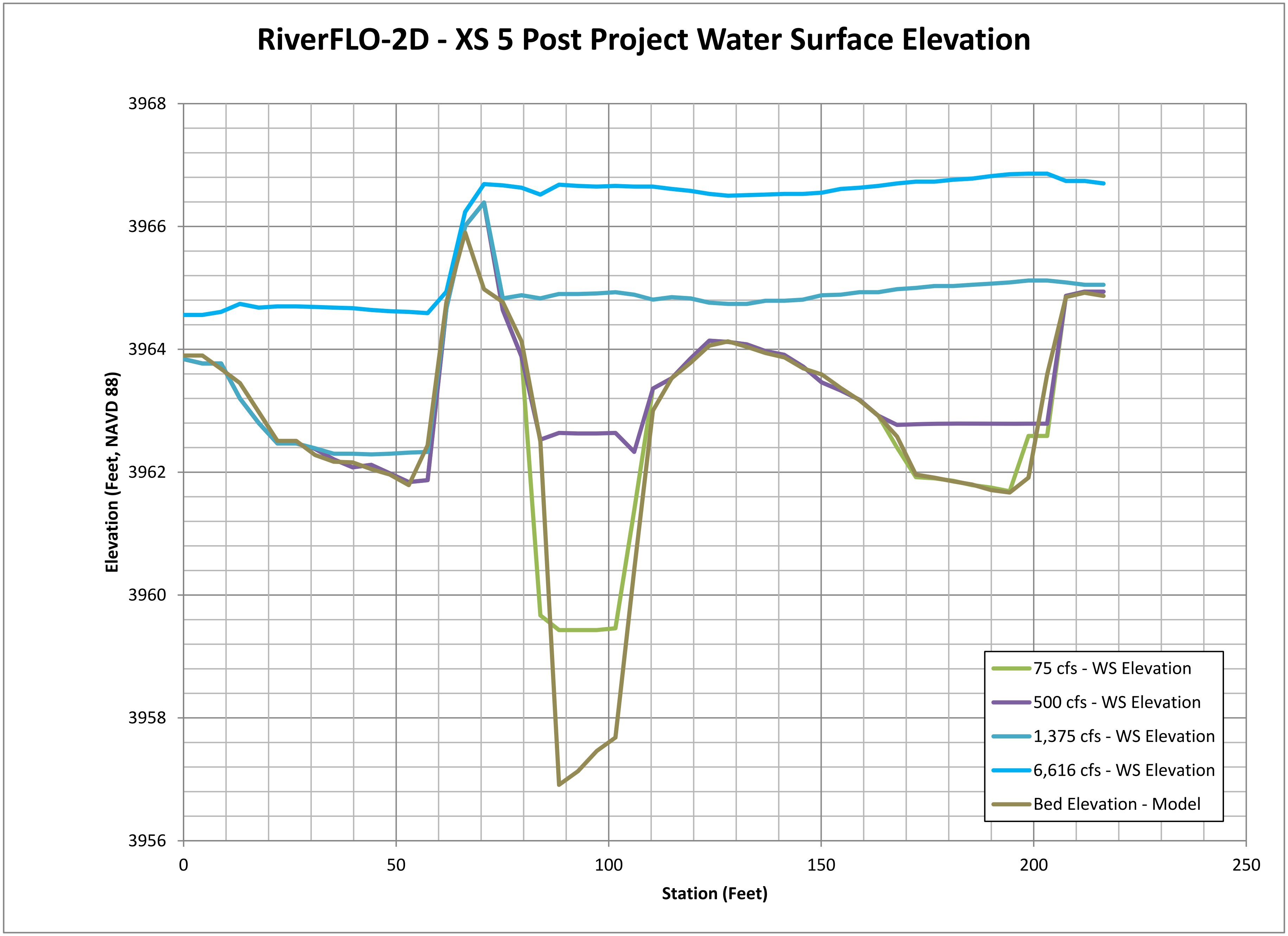
75 cfs - WS Elevation
500 cfs - WS Elevation
1375 cfs - WS Elevation
6616 cfs - WS Elevation
Existing Bed Elevation
Bed Elevation - Model


RiverFLO-2D - XS 1 Post Project Water Surface Elevation
75 cfs - WS Elevation
500 cfs - WS Elevation
1,375 cfs - WS Elevation
6,616 cfs - WS Elevation
Bed Elevation - Model
Elevation (Feet, NAVD 88)
3974
3972
3970
3968
3966
3964
3962
3960
3958
3956
3954
0
50
100
150
200
250
300
350
400
450
500
Station (Feet)

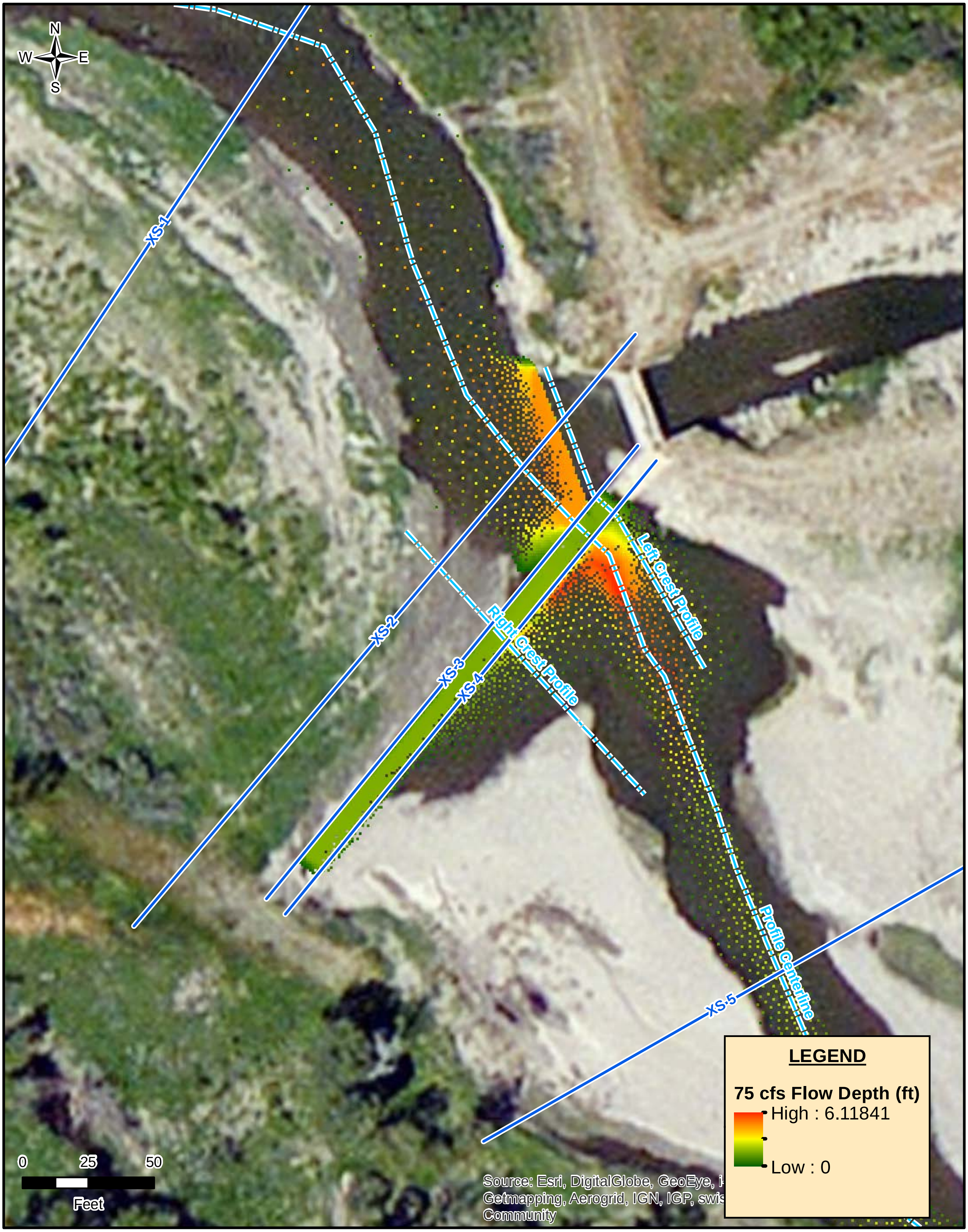

RiverFLO-2D - XS 2 Post Project Water Surface Elevation
Elevation (Feet, NAVD 88)
Station (Feet)
3974
3972
3970
3968
3966
3964
3962
3960
3958
3956
3954
0
50
100
150
200
250
300
350
75 cfs - WS Elevation
500 cfs - WS Elevation
1,375 cfs - WS Elevation
6,616 cfs - WS Elevation
Bed Elevation - Model
Existing Bed Elevation


RiverFLO-2D - XS 3 Post Project Water Surface Elevation
Elevation (Feet, NAVD 88)
3972
3970
3968
3966
3964
3962
3960
3958
3956
0
50
100
150
200
250
Station (Feet)
75 cfs - WS Elevation
500 cfs - WS Elevation
1,375 cfs - WS Elevation
6,616 cfs - WS Elevation
Bed Elevation - Model
Existing Bed Elevation


RiverFLO-2D - XS 4 Post Project Water Surface Elevation
Elevation (Feet, NAVD 88)
3972
3970
3968
3966
3964
3962
3960
3958
3956
3954
0
50
100
150
200
250
Station (Feet)
75 cfs - WS Elevation
500 cfs - WS Elevation
1,375 cfs - WS Elevation
6,616 cfs - WS Elevation
Bed Elevation - Model
Existing Bed Elevation



RiverFLO-2D - XS 5 Post Project Water Surface Elevation
3968
3966
3964
3962
3960
3958
3956
Elevation (Feet, NAVD 88)
0
50
100
150
200
250
Station (Feet)
75 cfs - WS Elevation
500 cfs - WS Elevation
1,375 cfs - WS Elevation
6,616 cfs - WS Elevation
Bed Elevation - Model



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DEADMAN'S BASIN DIVERSION DAM AND HEADGATE REPLACEMENT PROJECT

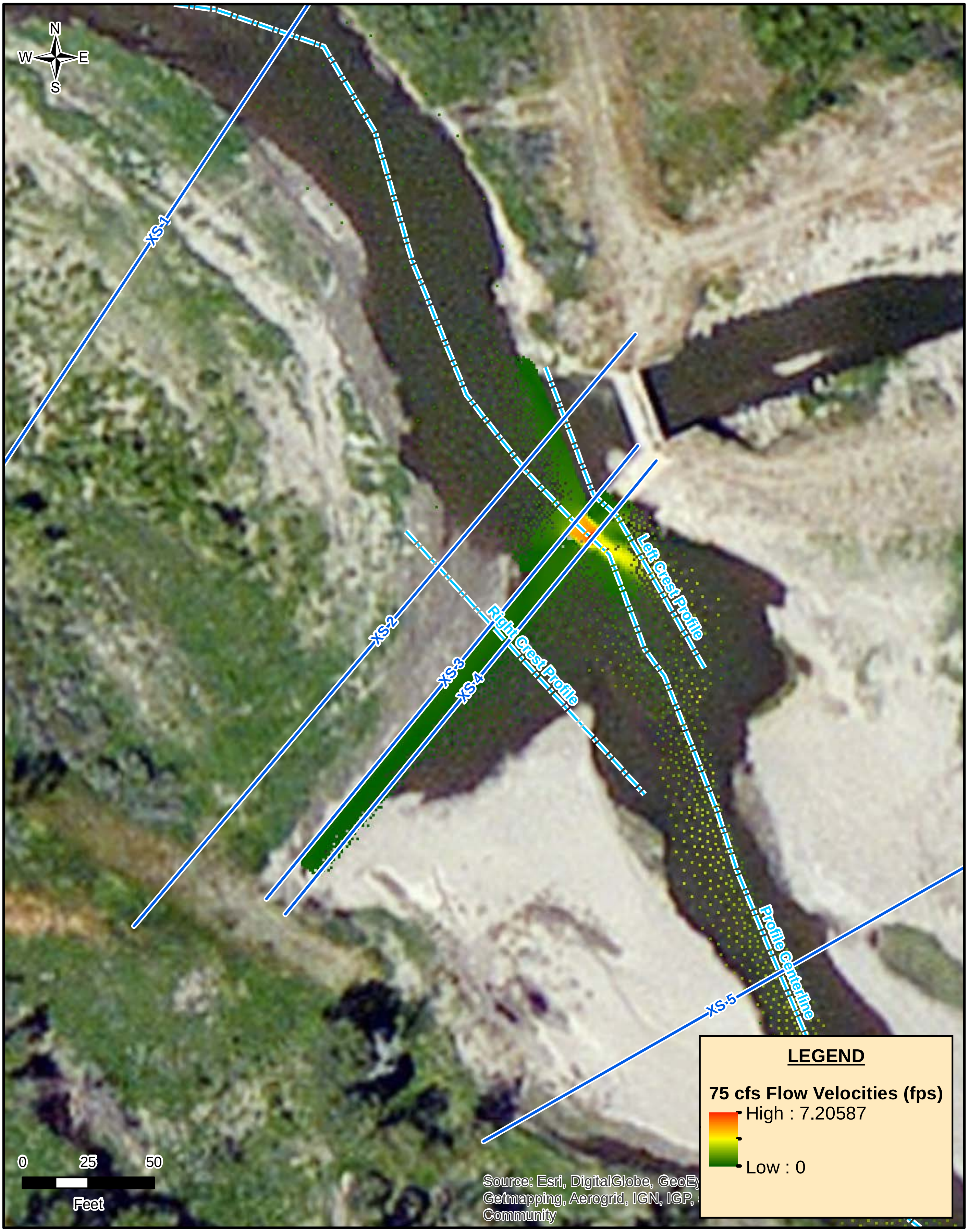
WHEATLAND COUNTY MONTANA

PROJECT NO. 1447.035

Existing Conditions - 75 cfs Flow Depth

FIGURE NO. A

M:\1447\035\GIS\Exhibits\EX 75cfs Depth.mxd



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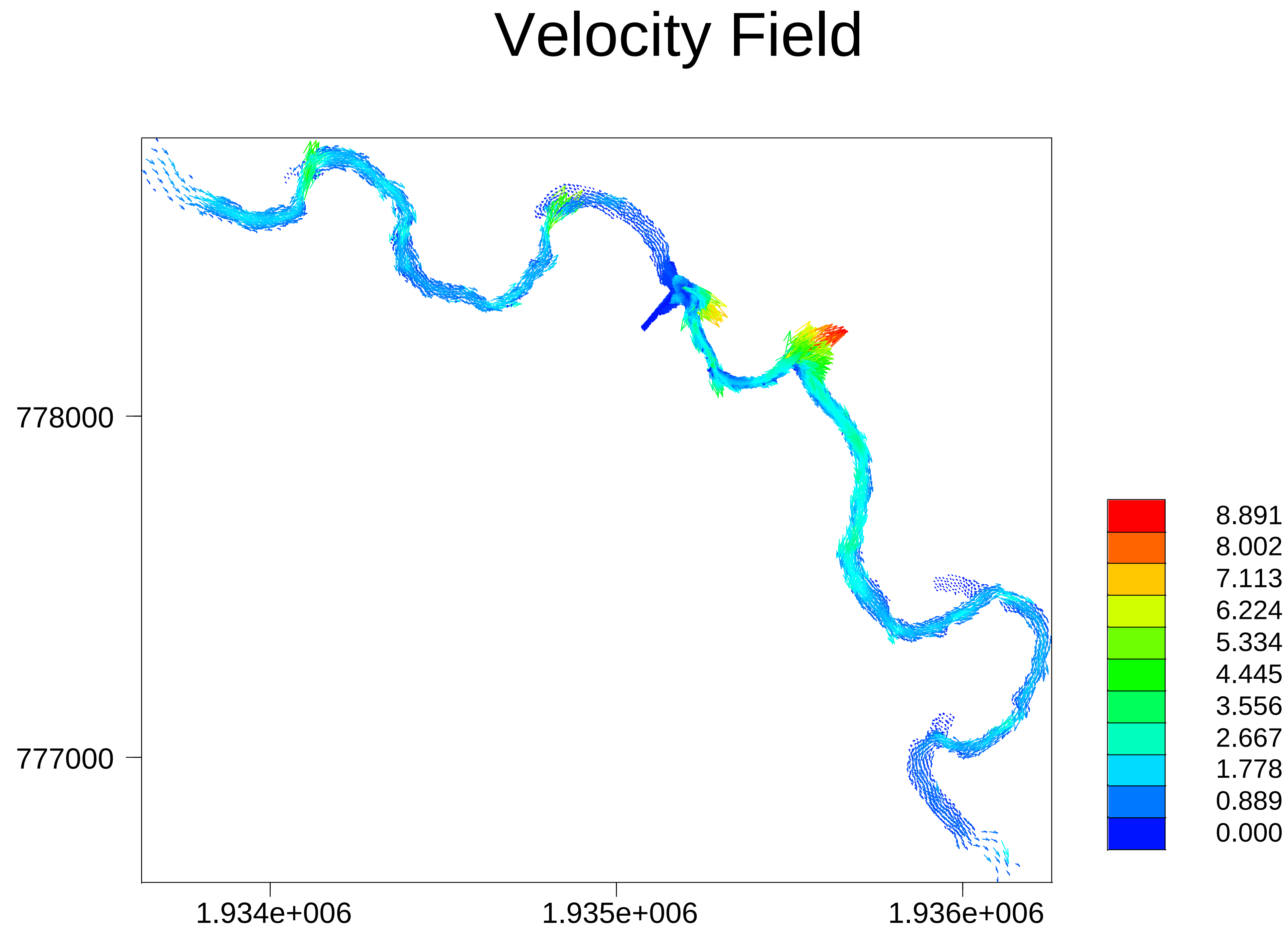
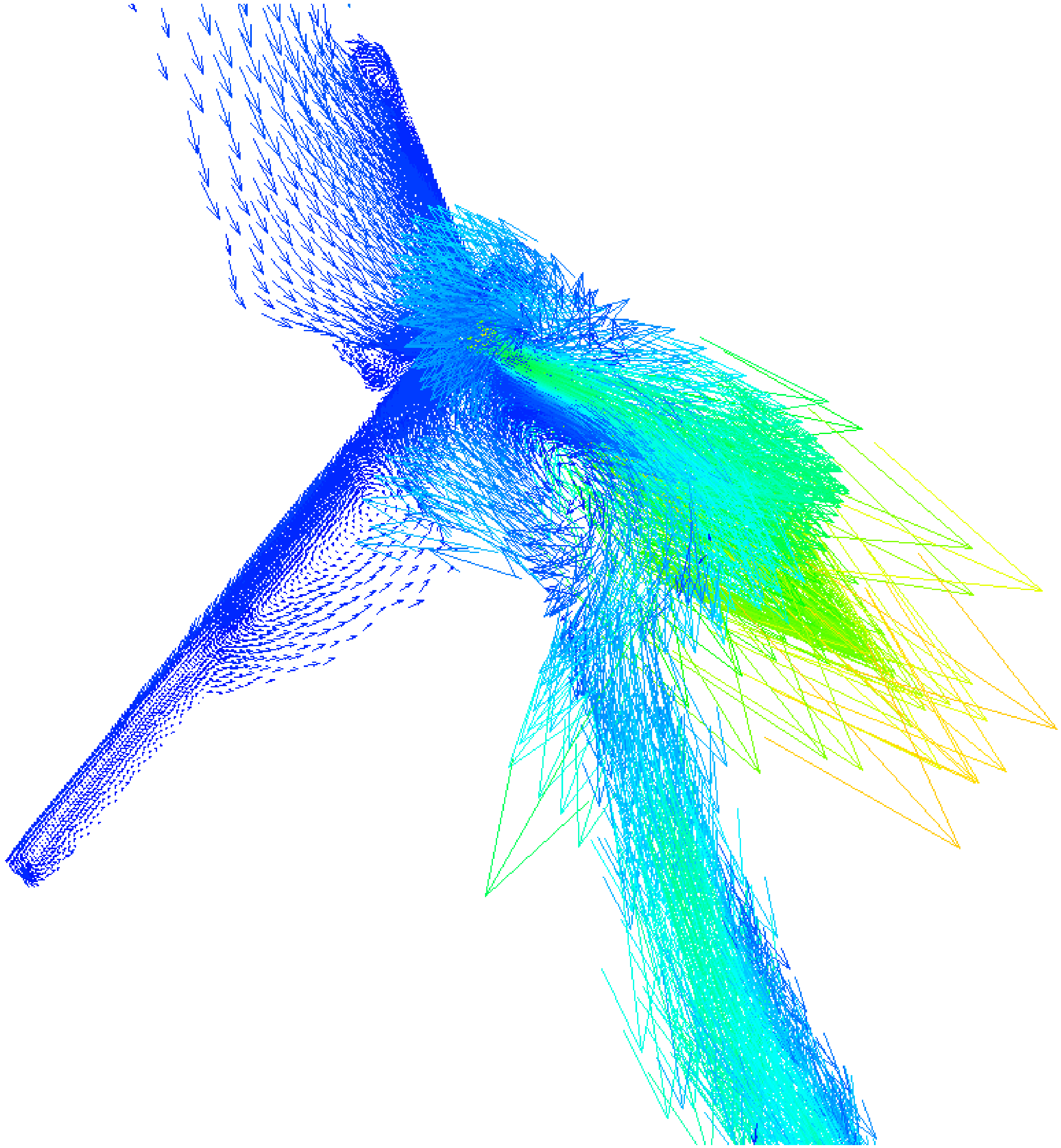
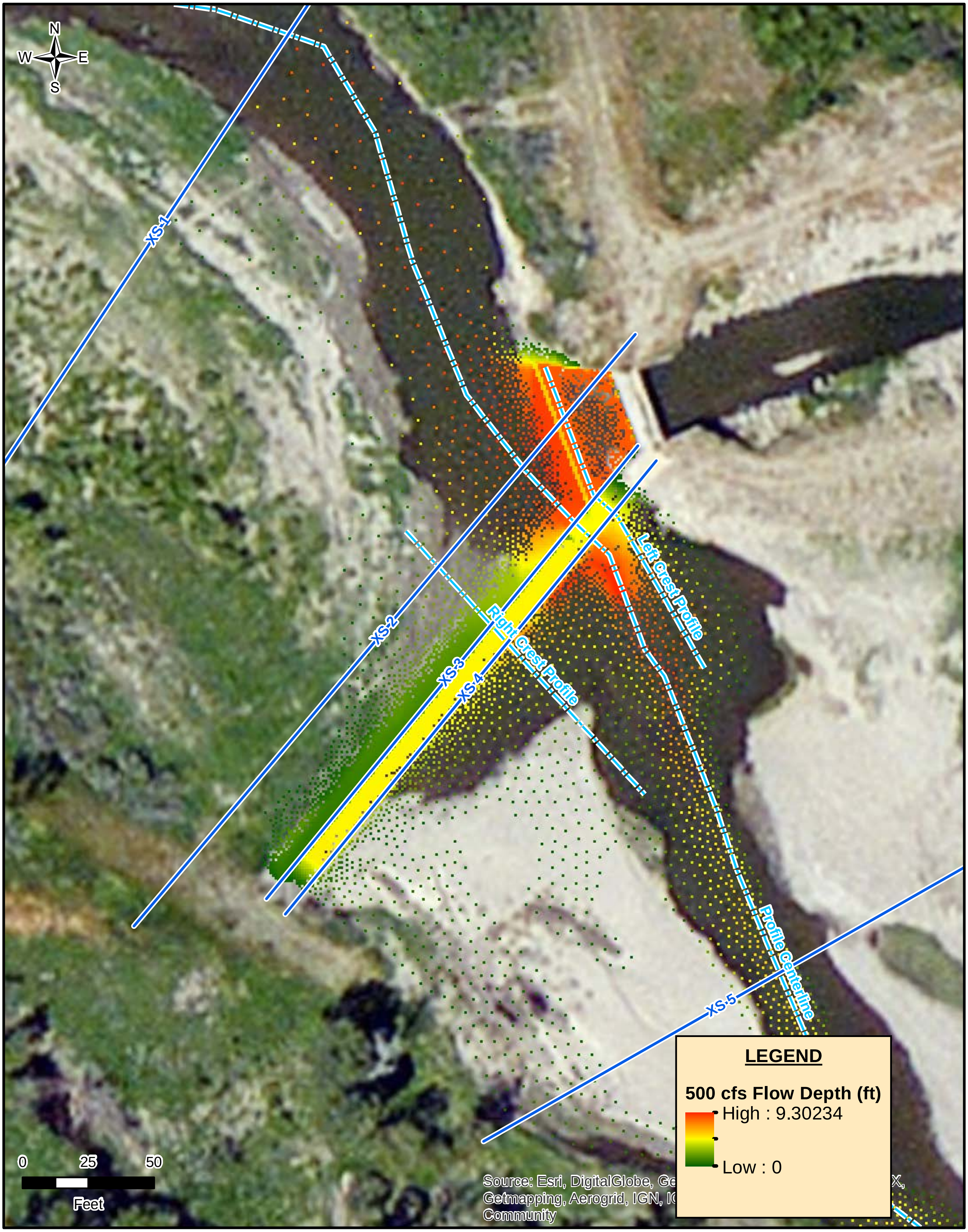
		Existing Conditions - 75 cfs Flow Velocities	FIGURE NO. A

M:\1447\035\GIS\Exhibits\EX 75cfs Velocity.mxd



Velocity Field
778000
777000
1.934e+006
1.935e+006
1.936e+006
8.891
8.002
7.113
6.224
5.334
4.445
3.556
2.667
1.778
0.889
0.000





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DEADMAN'S BASIN DIVERSION DAM AND HEADGATE REPLACEMENT PROJECT

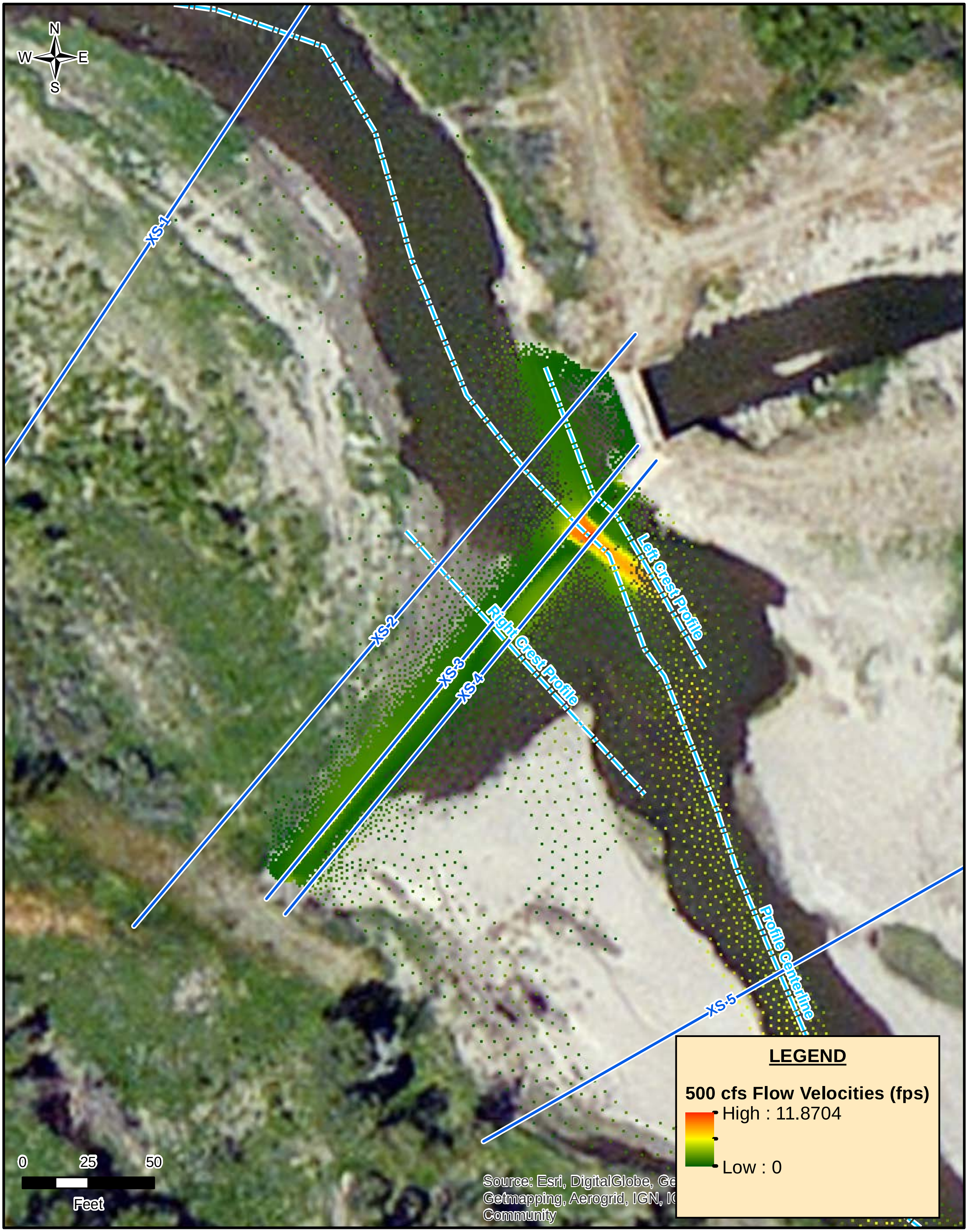
WHEATLAND COUNTY MONTANA

PROJECT NO. 1447.035

FIGURE NO. A

Existing Conditions - 500 cfs Flow Depth

M:\1447\035\GIS\Exhibits\EX 500cfs Depth.mxd



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DEADMAN'S BASIN DIVERSION DAM AND HEADGATE REPLACEMENT PROJECT

WHEATLAND COUNTY MONTANA

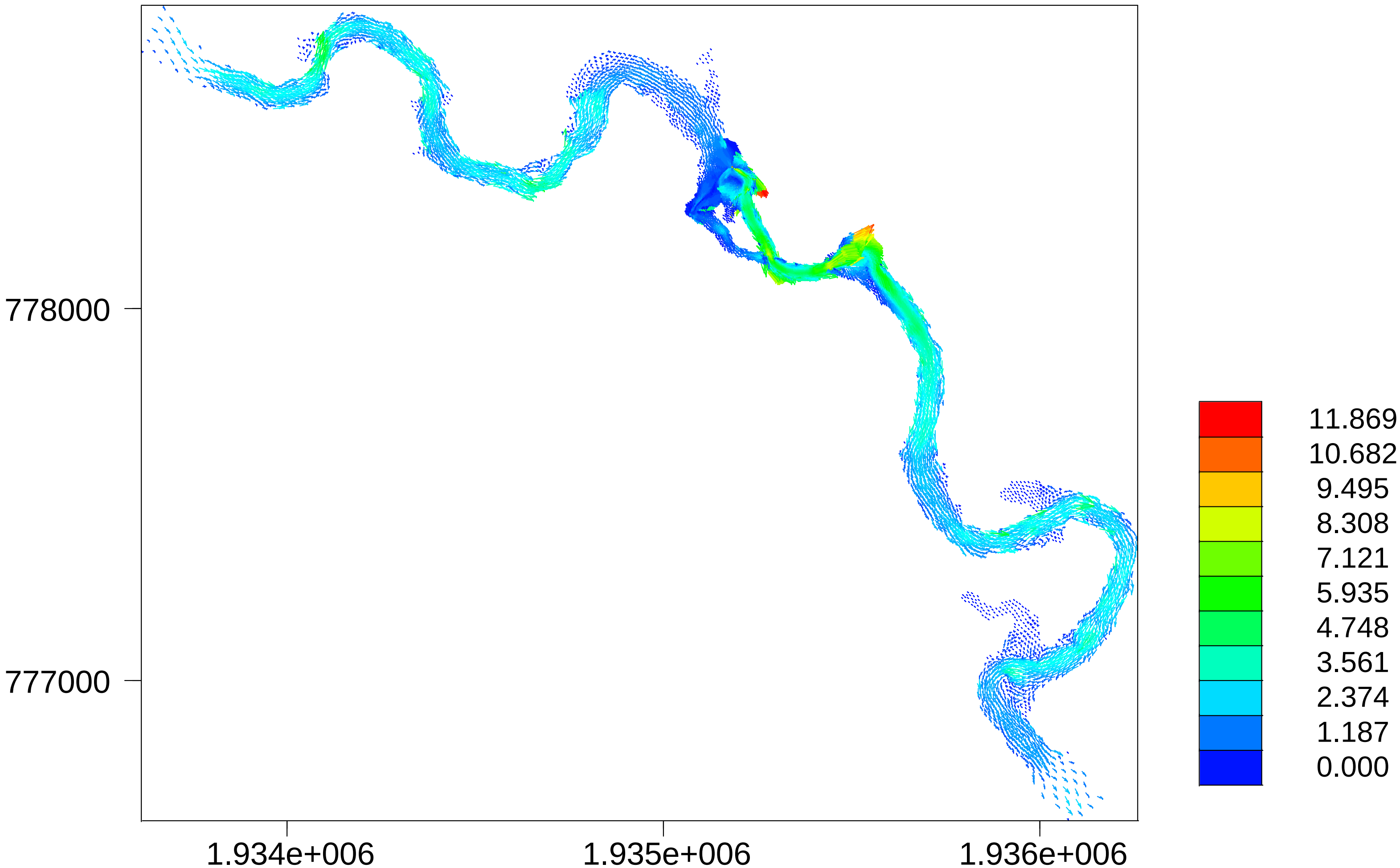
Existing Conditions - 500 cfs Flow Velocities

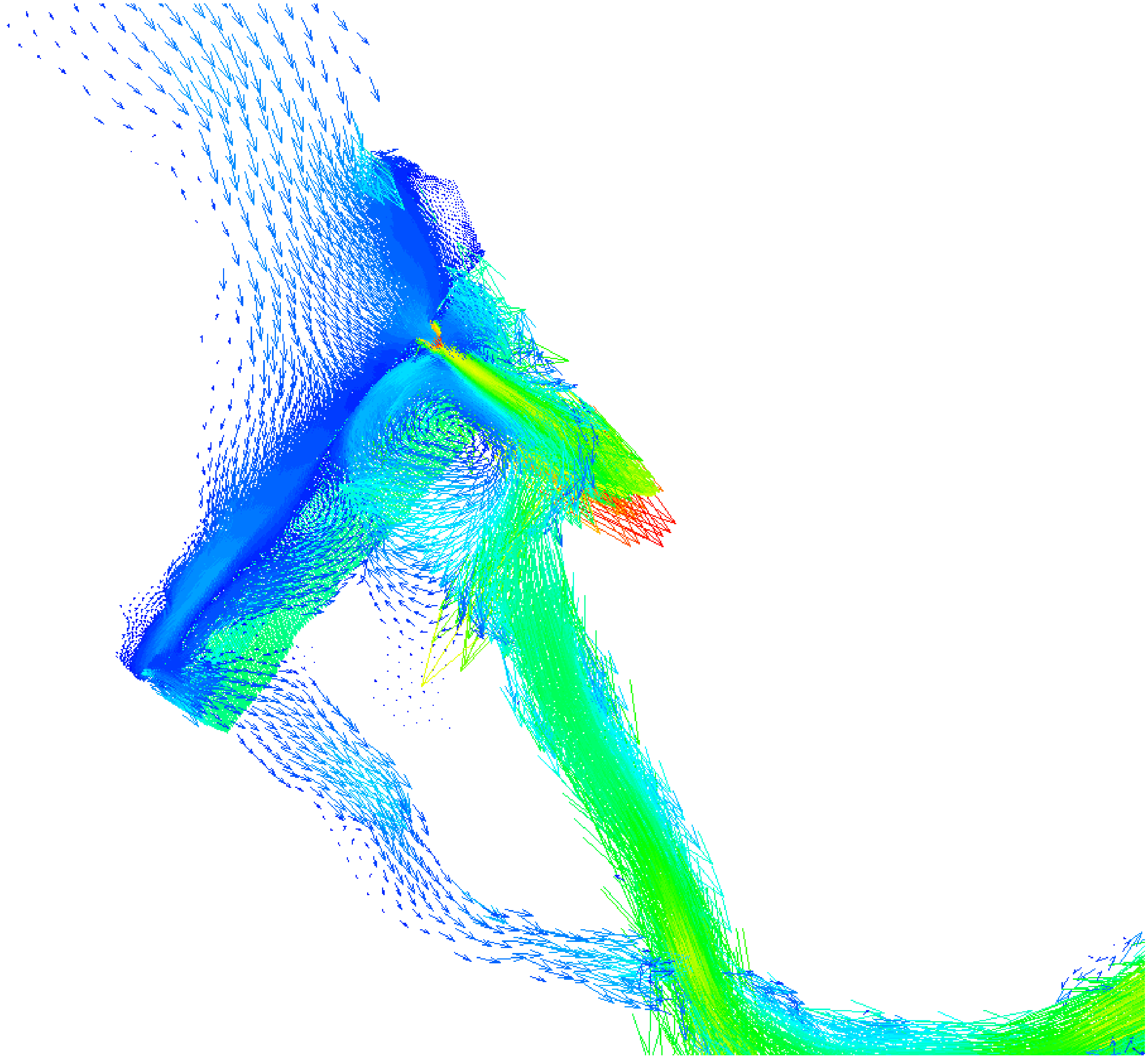
PROJECT NO. 1447.035

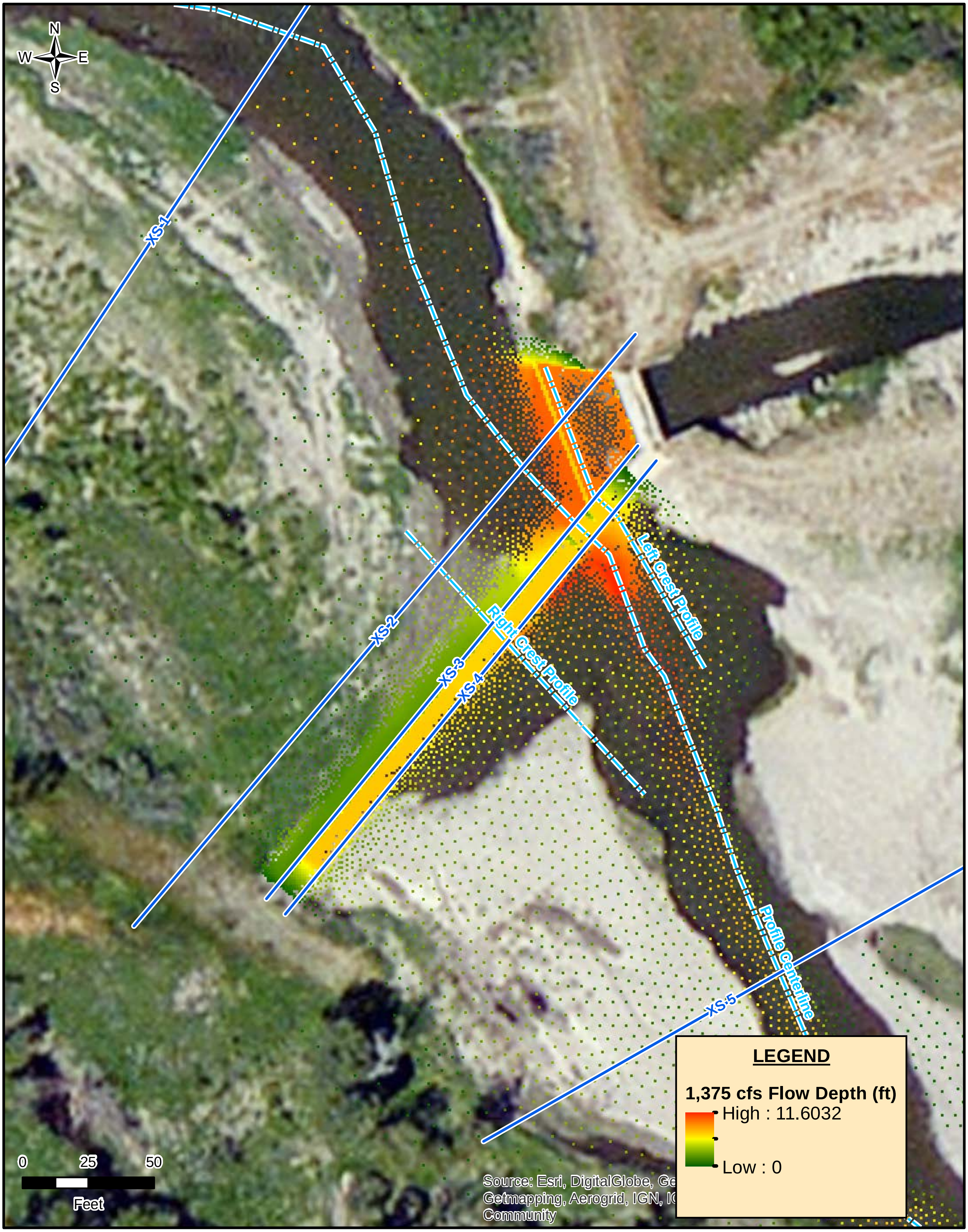
FIGURE NO. A

M:\1447\035\GIS\Exhibits\EX 500cfs Velocity.mxd

Velocity Field







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DEADMAN'S BASIN DIVERSION DAM AND HEADGATE REPLACEMENT PROJECT

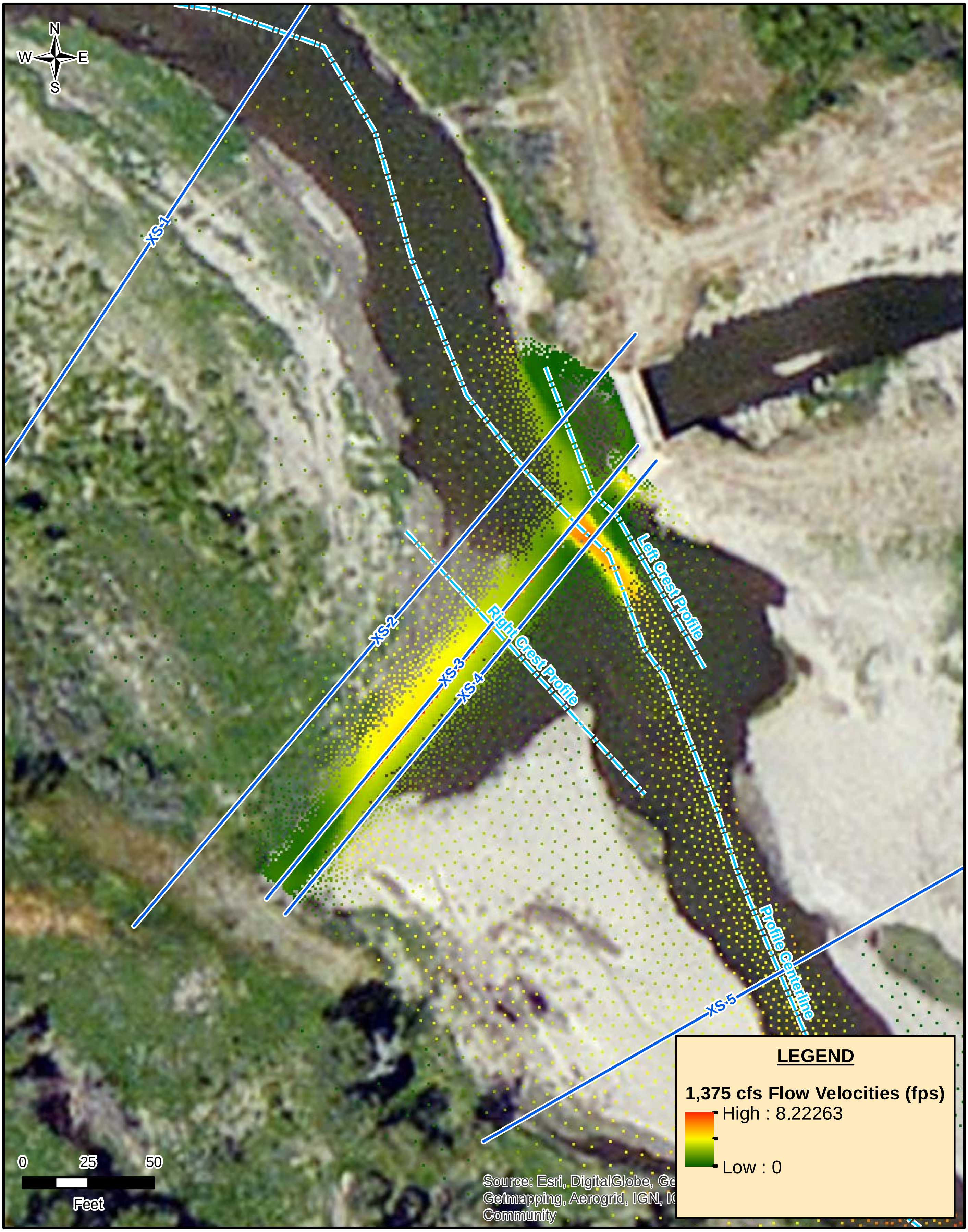
WHEATLAND COUNTY MONTANA

PROJECT NO. 1447.035

Existing Conditions - 1,375 cfs (2-yr) Flow Depth

FIGURE NO. A

M:\1447\035\GIS\Exhibits\EX 1375cfs Depth.mxd



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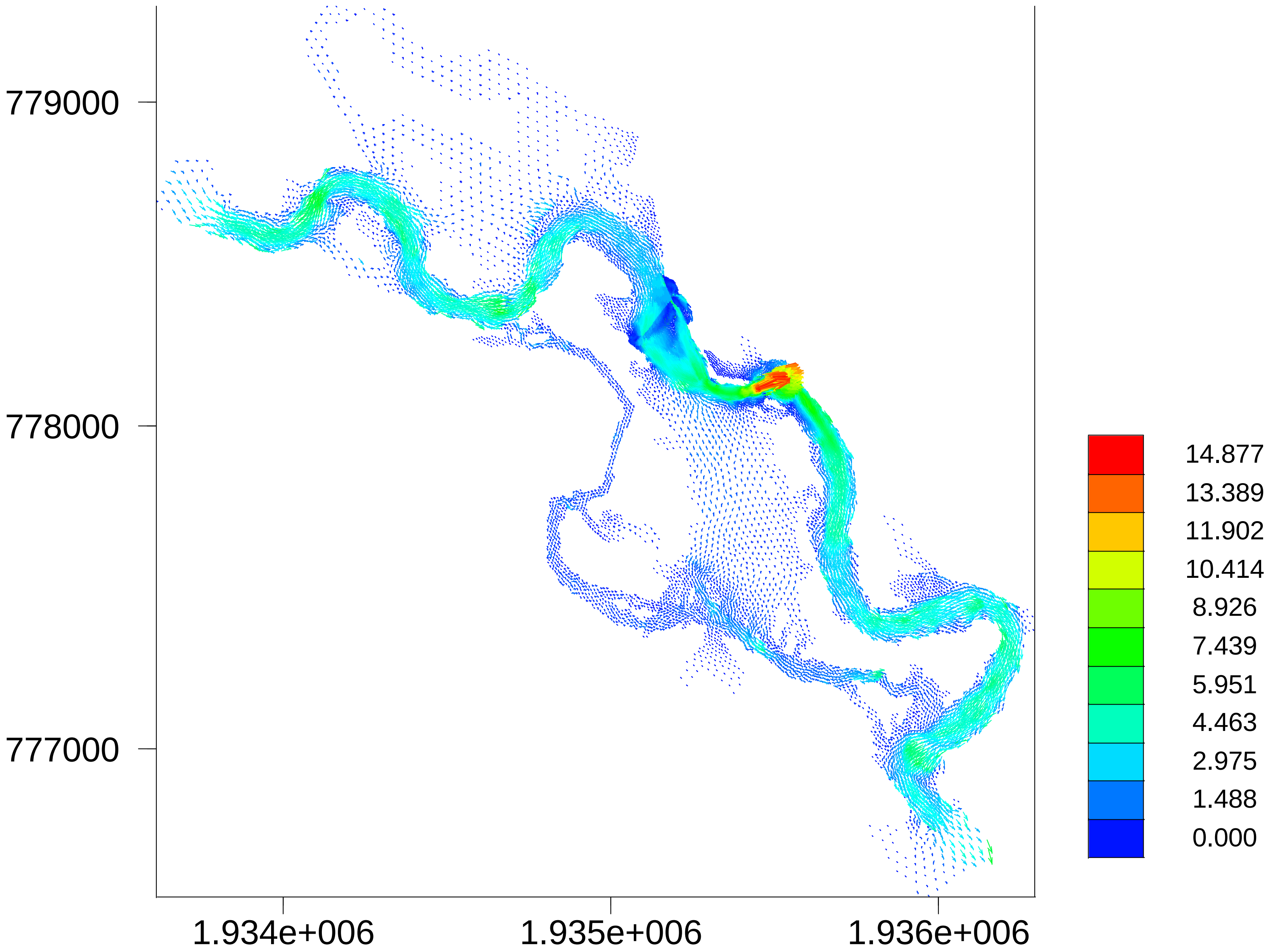
WHEATLAND COUNTY MONTANA

Existing Conditions - 1,375 cfs (2-yr) Flow Velocities

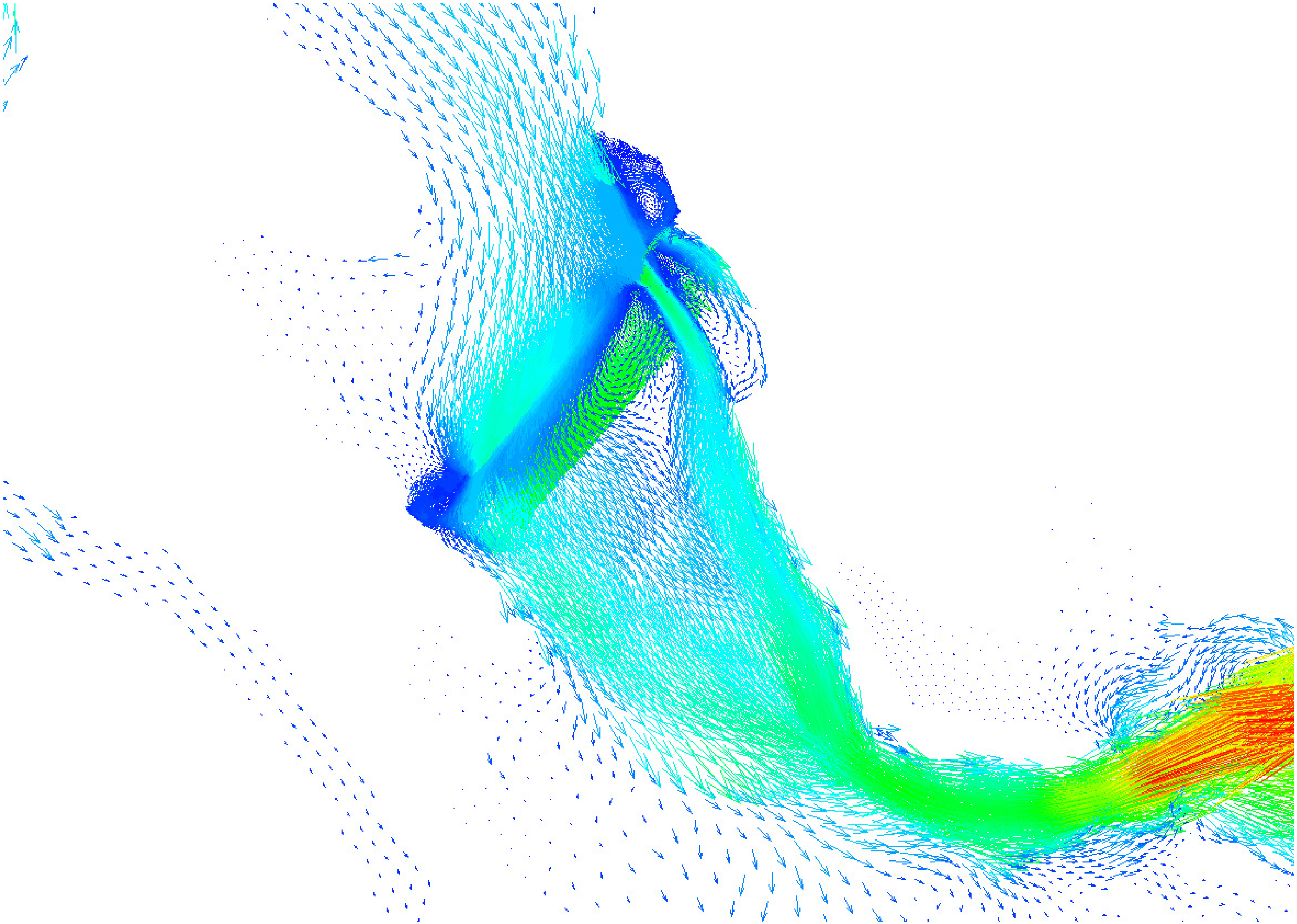
PROJECT NO. 1447.035

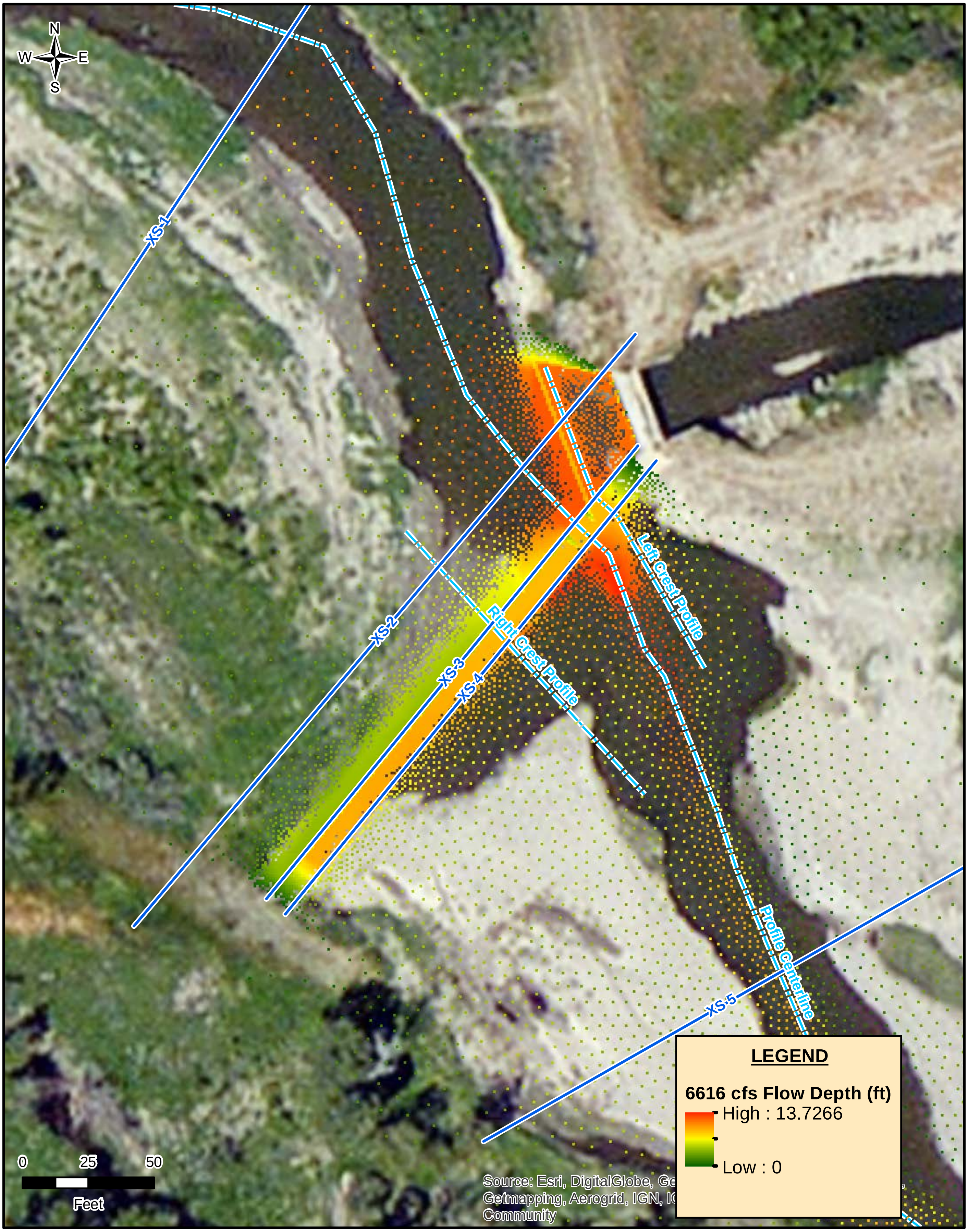
FIGURE NO. A

M:\1447\035\GIS\Exhibits\EX 1375cfs Velocity.mxd



779000
778000
777000
1.934e+006
1.935e+006
1.936e+006
14.877
13.389
11.902
10.414
8.926
7.439
5.951
4.463
2.975
1.488
0.000





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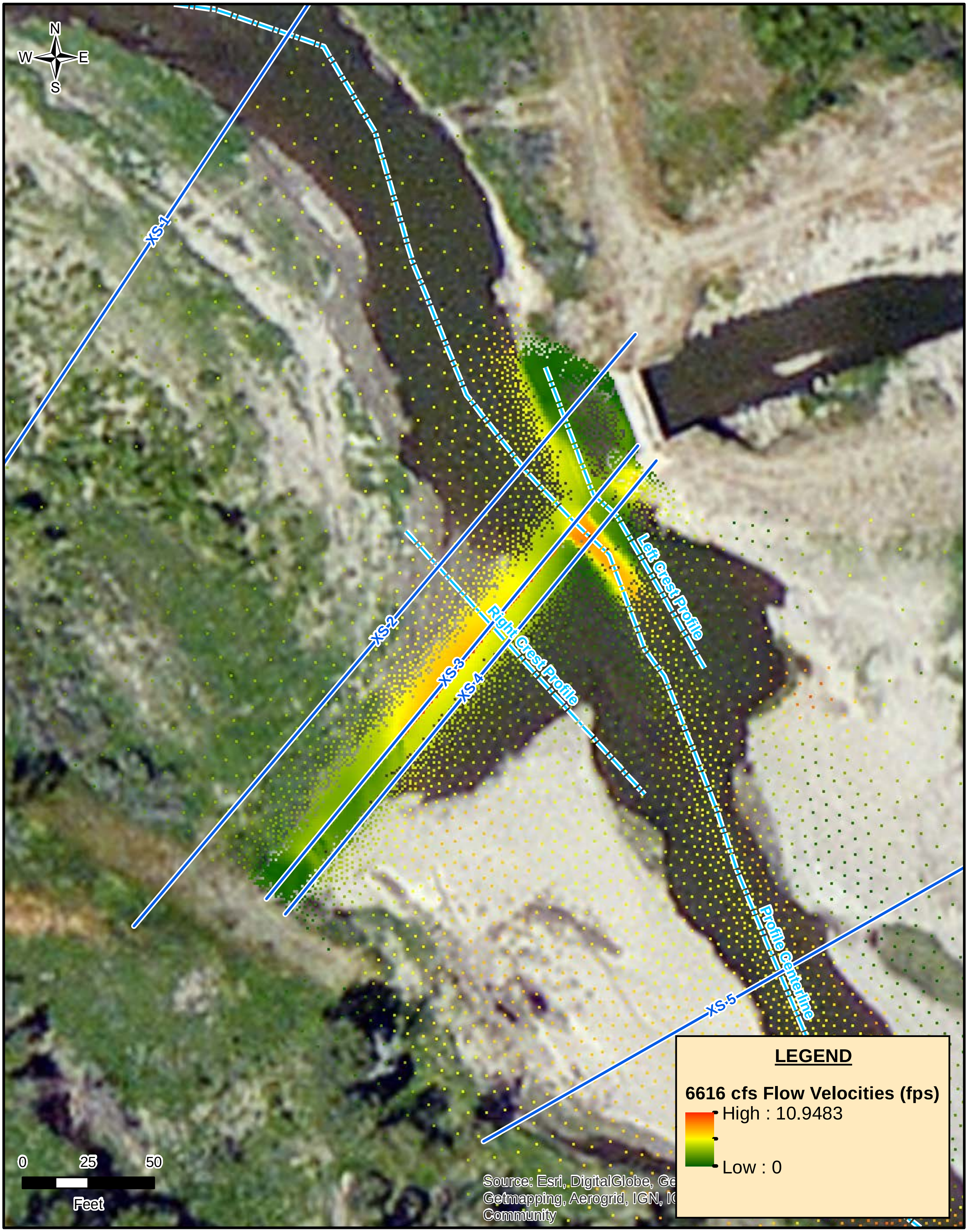
DEADMAN'S BASIN DIVERSION DAM AND
HEADGATE REPLACEMENT PROJECT
WHEATLAND COUNTY
MONTANA

PROJECT NO.
1447.035

Existing Conditions - 6,616 cfs (100-yr)
Flow Depth

FIGURE NO.
A

M:\1447\035\GIS\Exhibits\EX 6616cfs Depth.mxd



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DEADMAN'S BASIN DIVERSION DAM AND HEADGATE REPLACEMENT PROJECT

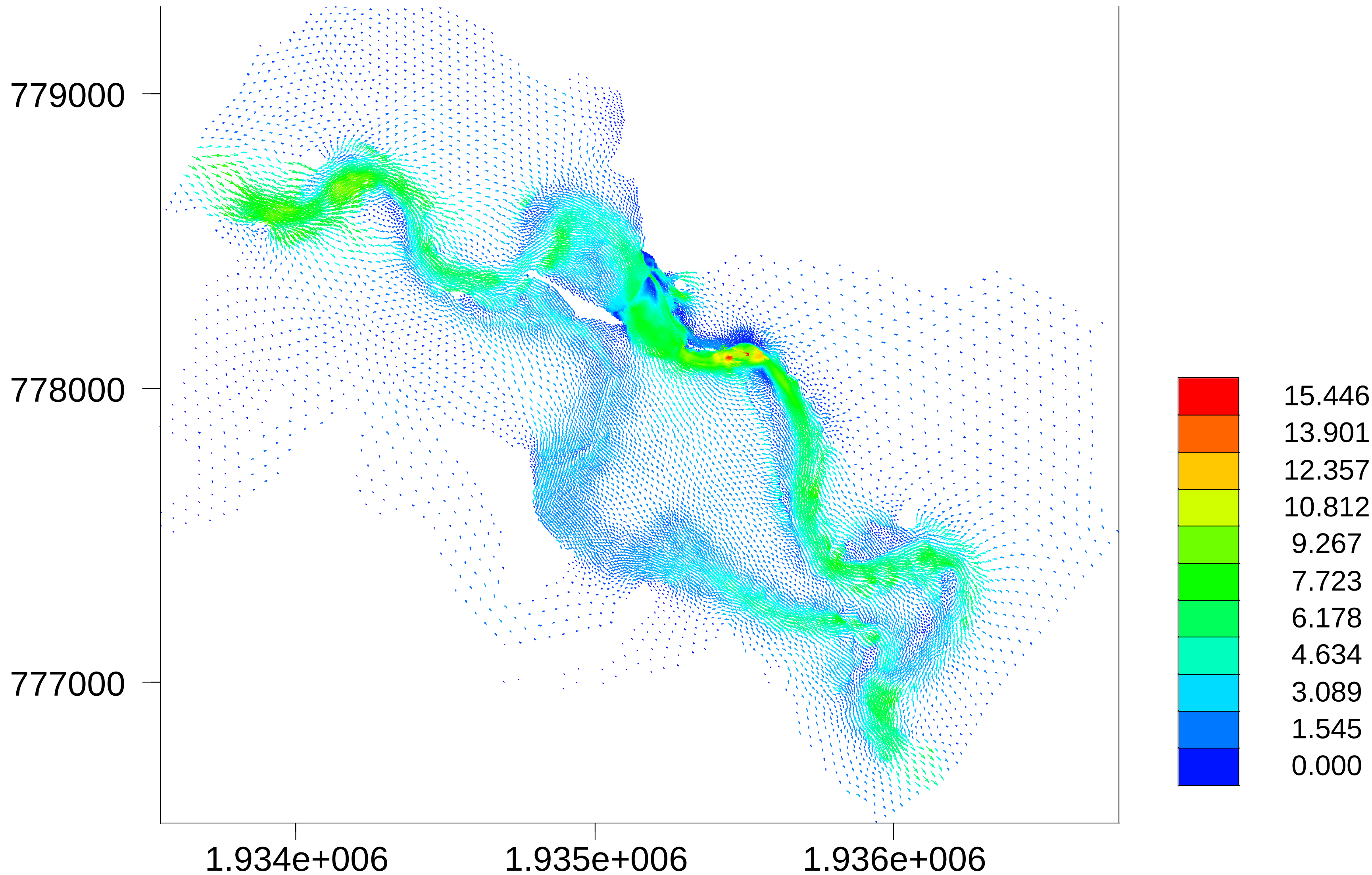
WHEATLAND COUNTY MONTANA

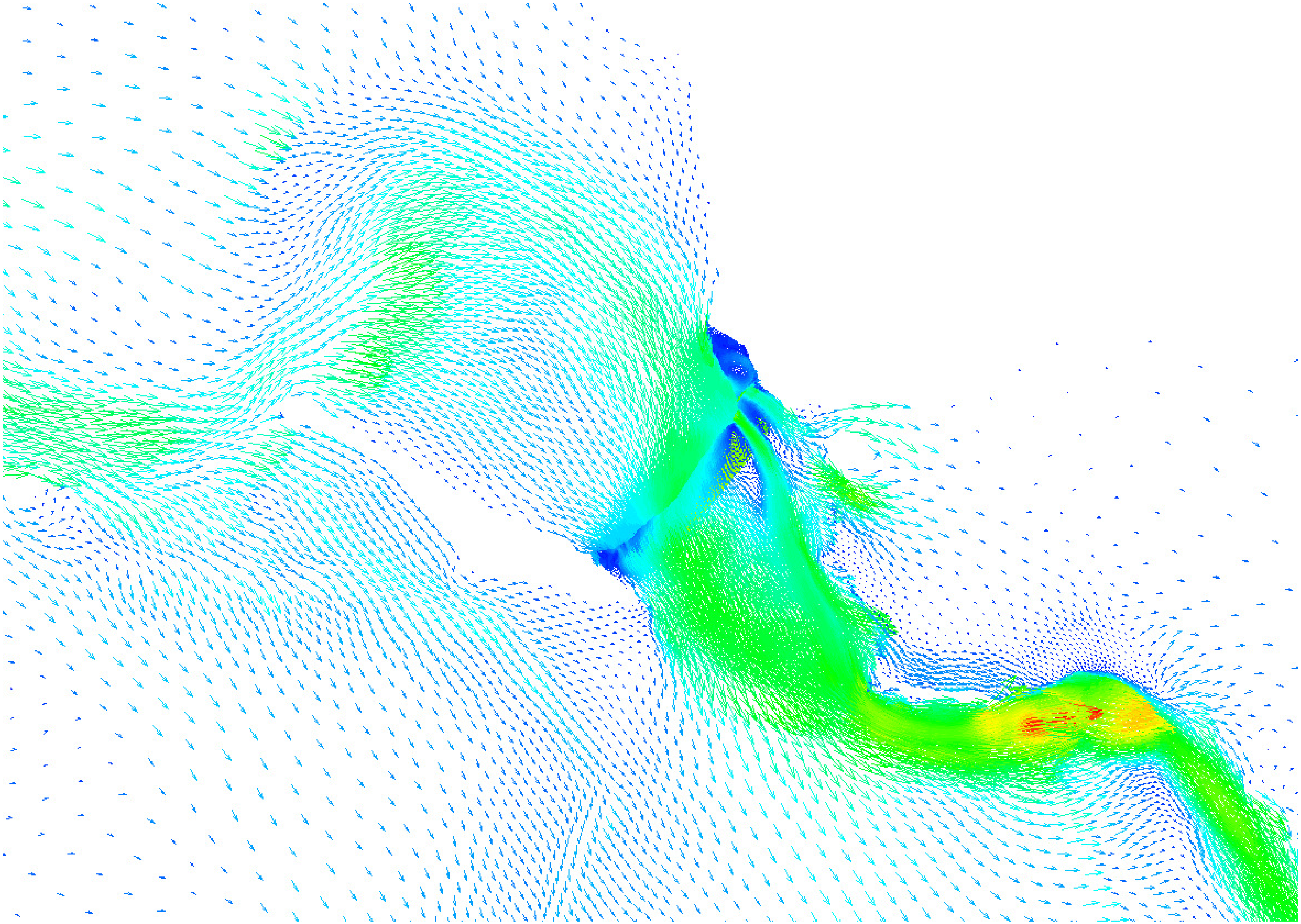
PROJECT NO. 1447.035

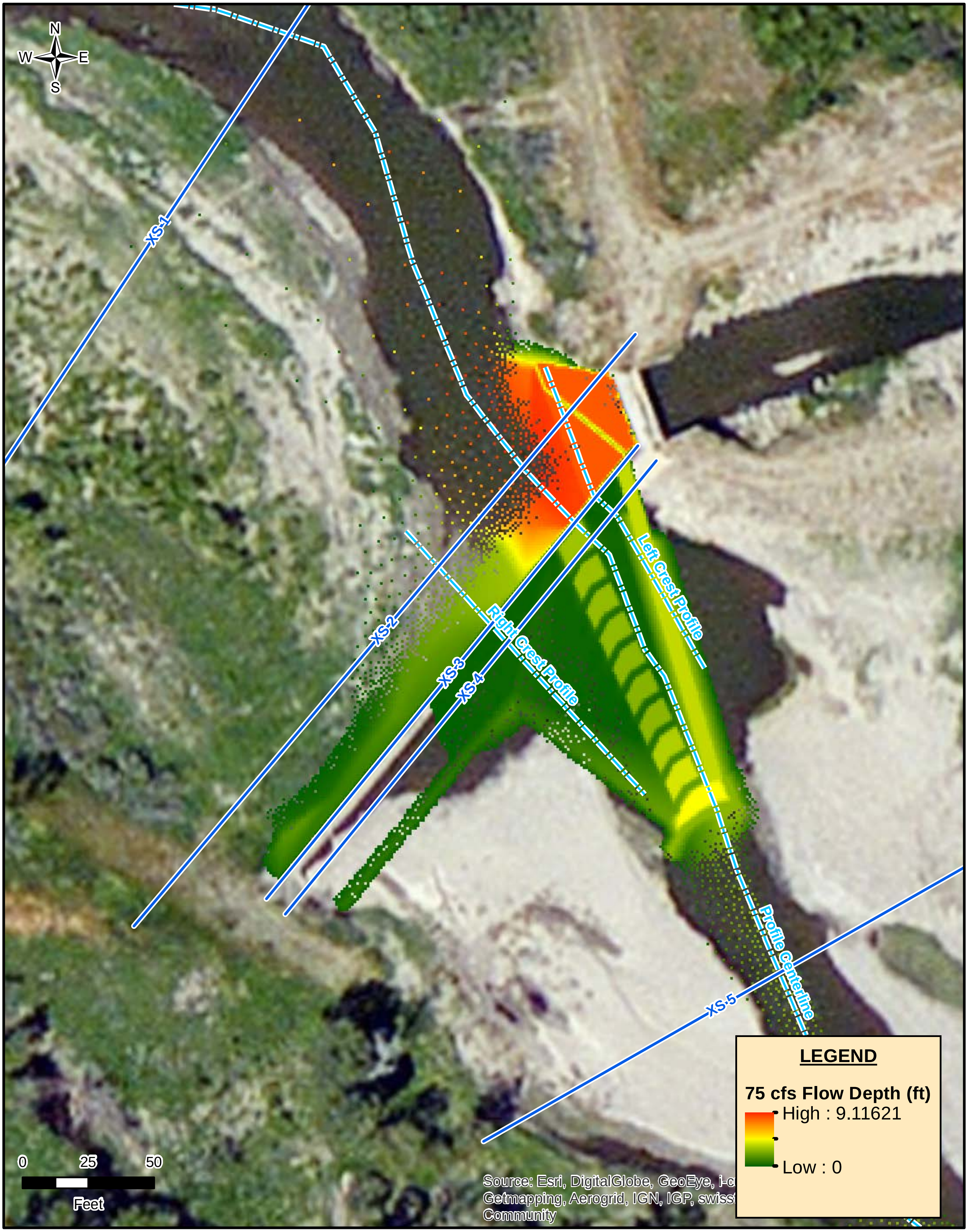
Existing Conditions - 6,616 cfs (100-yr) Flow Velocities

FIGURE NO. A

M:\1447\035\GIS\Exhibits\EX 6616cfs Velocity.mxd


779000
778000
777000
1.934e+006
1.935e+006
1.936e+006
15.446
13.901
12.357
10.812
9.267
7.723
6.178
4.634
3.089
1.545
0.000





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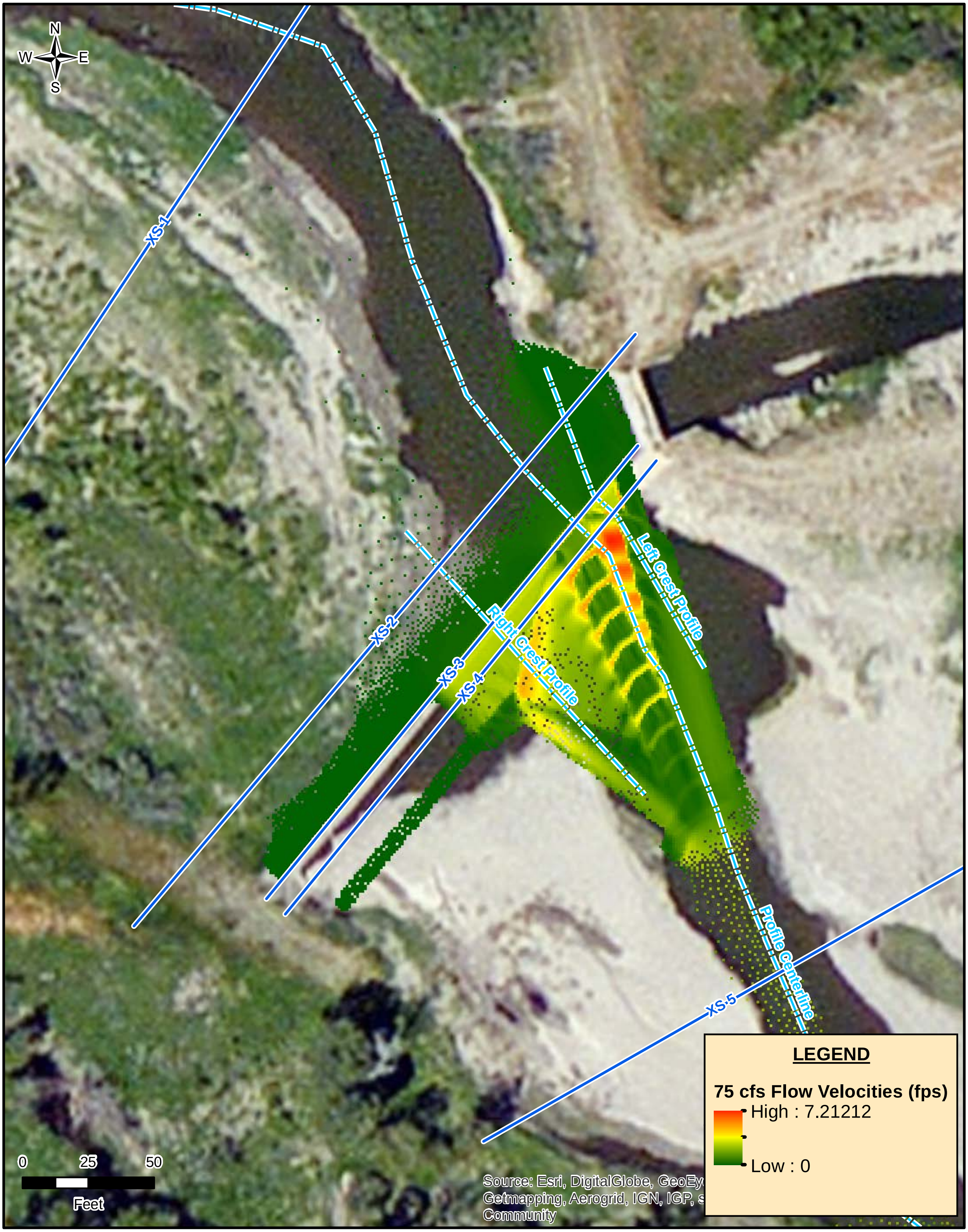
DEADMAN'S BASIN DIVERSION DAM AND HEADGATE REPLACEMENT PROJECT
WHEATLAND COUNTY MONTANA

Post-Project - 75 cfs Flow Depth

PROJECT NO. 1447.035

FIGURE NO. A

M:\1447\035\GIS\Exhibits\PP 75cfs Depth.mxd



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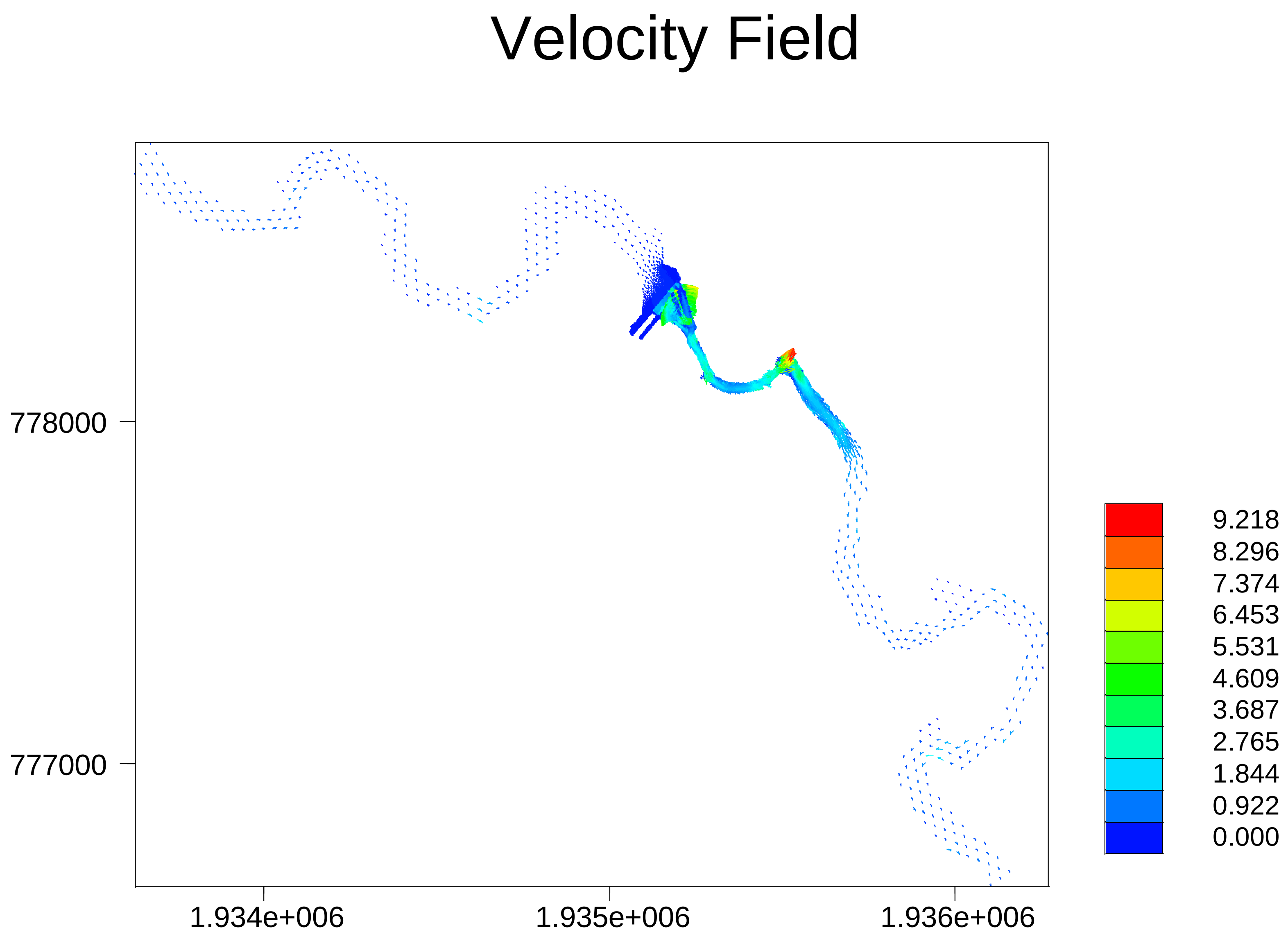
WHEATLAND COUNTY MONTANA

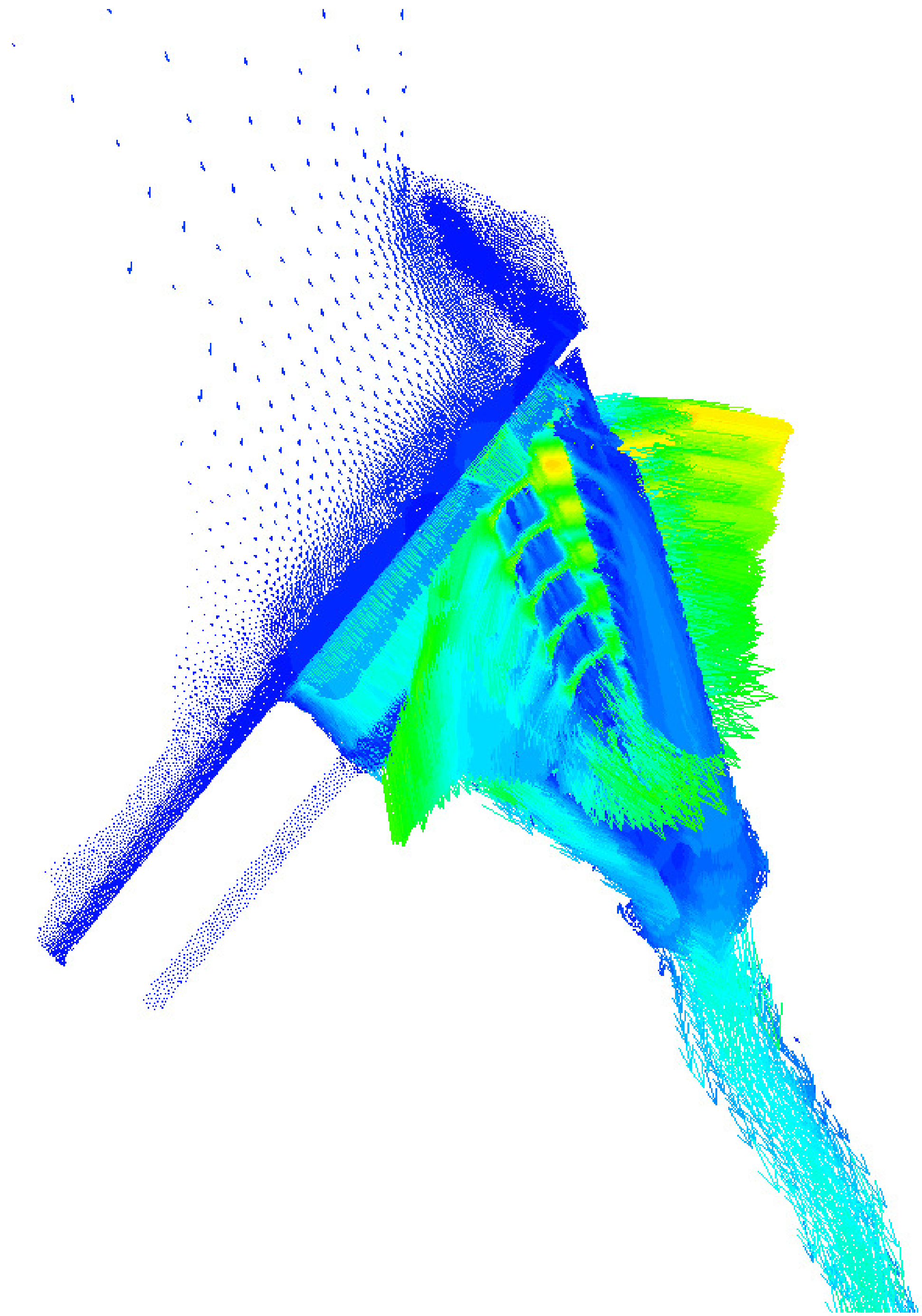
PROJECT NO. 1447.035

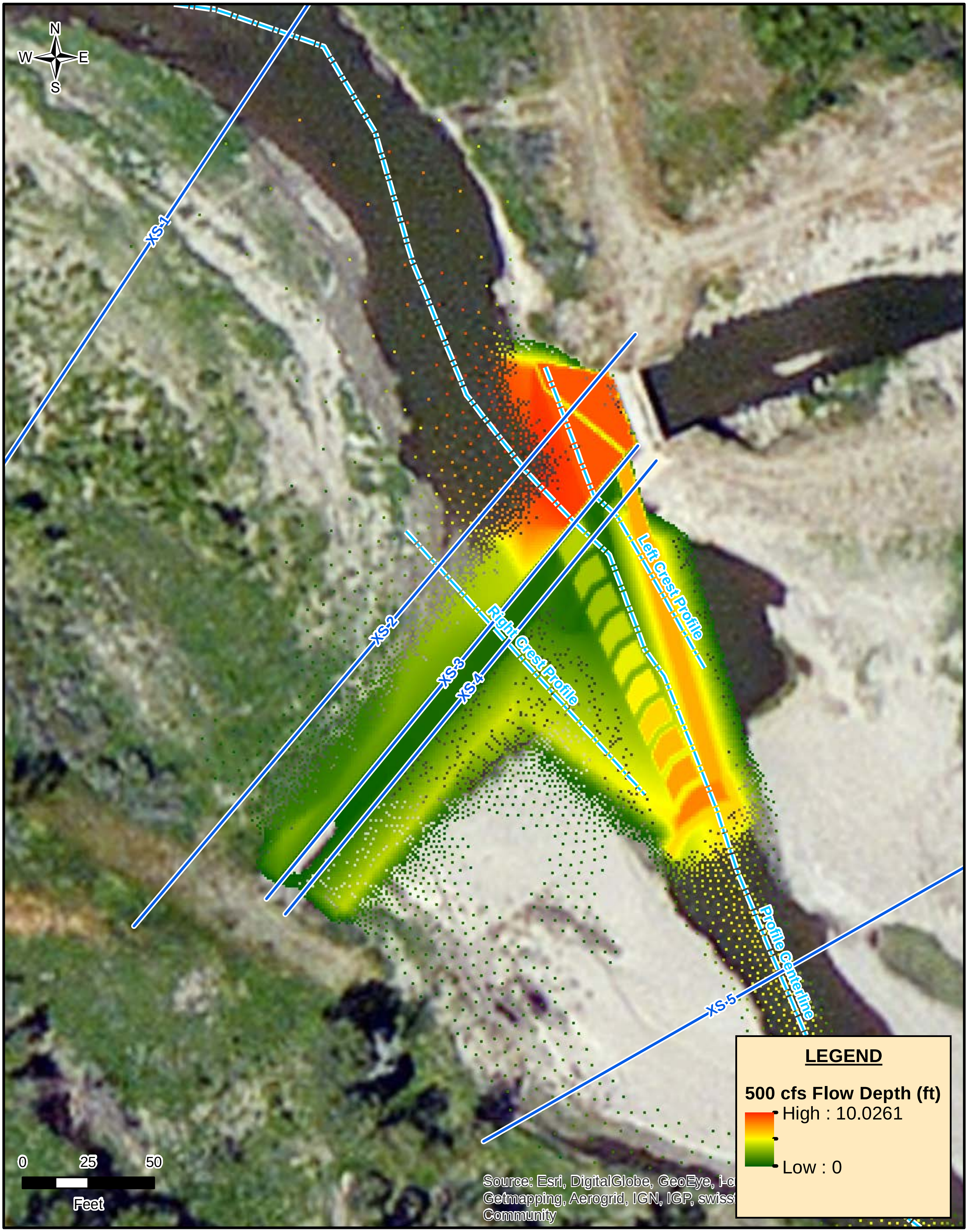
Post-Project - 75 cfs Flow Velocities

FIGURE NO. A

M:\1447\035\GIS\Exhibits\PP 75cfs Velocity.mxd


Velocity Field
778000
777000
1.934e+006
1.935e+006
1.936e+006
9.218
8.296
7.374
6.453
5.531
4.609
3.687
2.765
1.844
0.922
0.000





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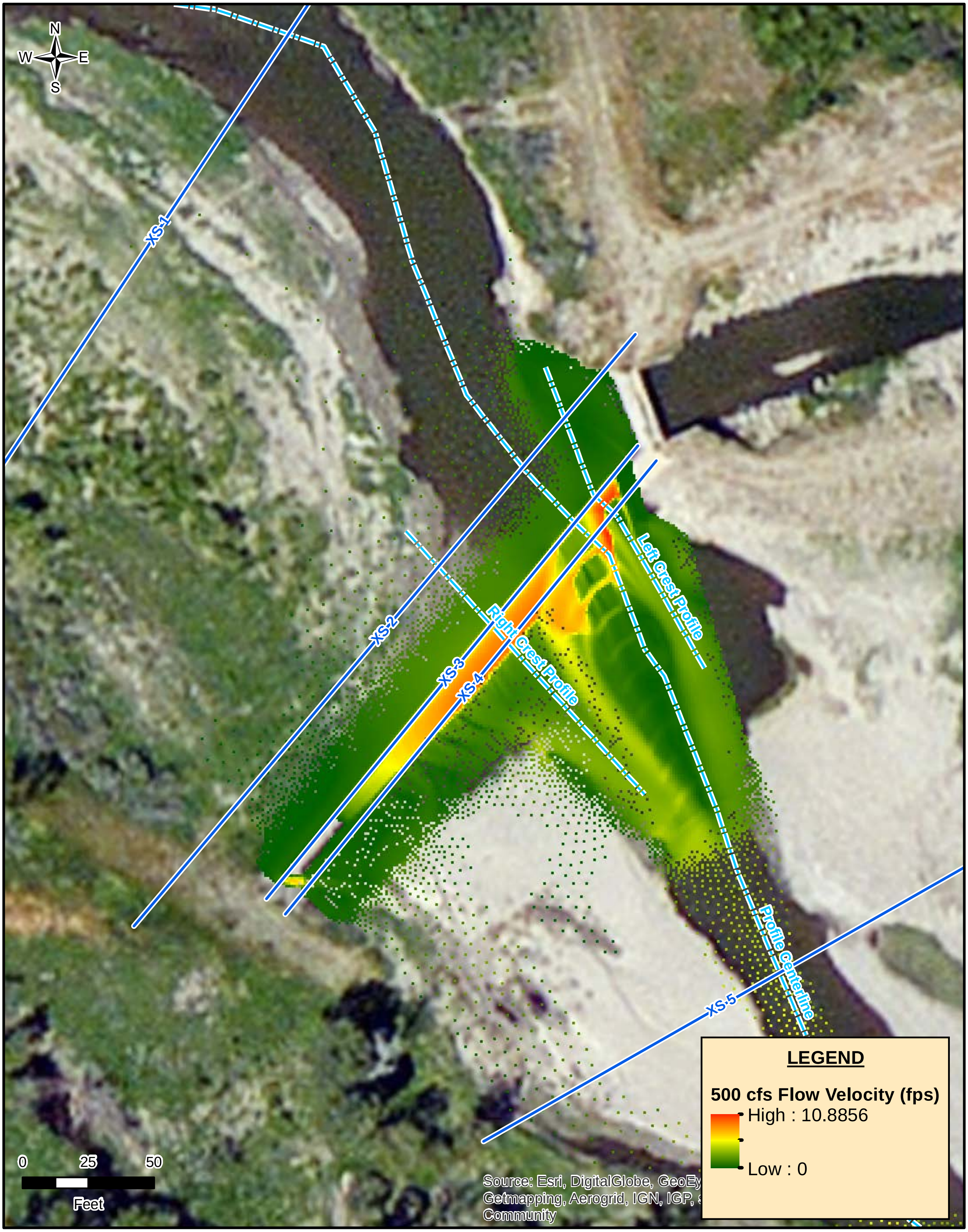
		Post-Project - 500 cfs Flow Depth	FIGURE NO. A	

M:\1447\035\GIS\Exhibits\PP 500cfs Depth.mxd



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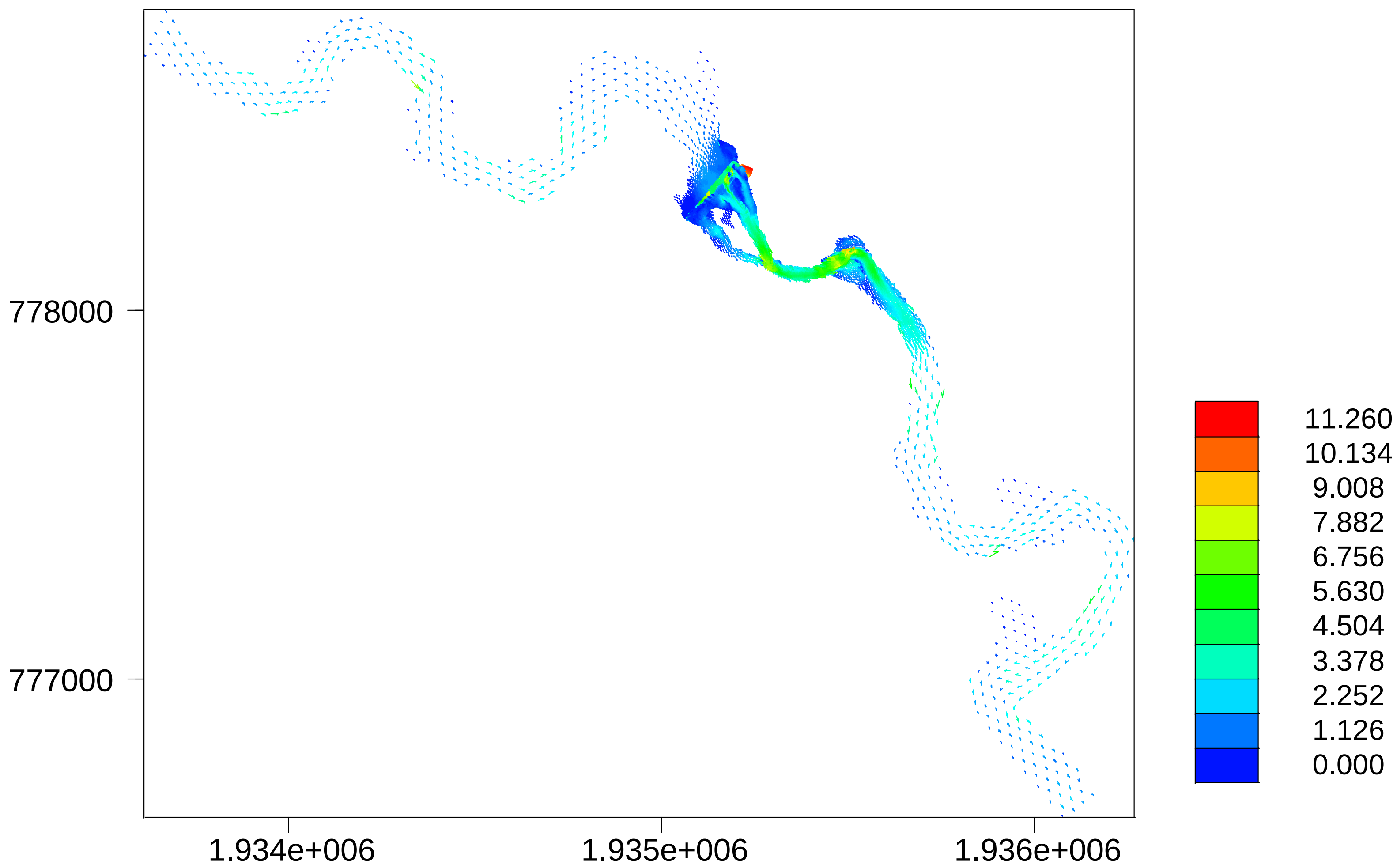
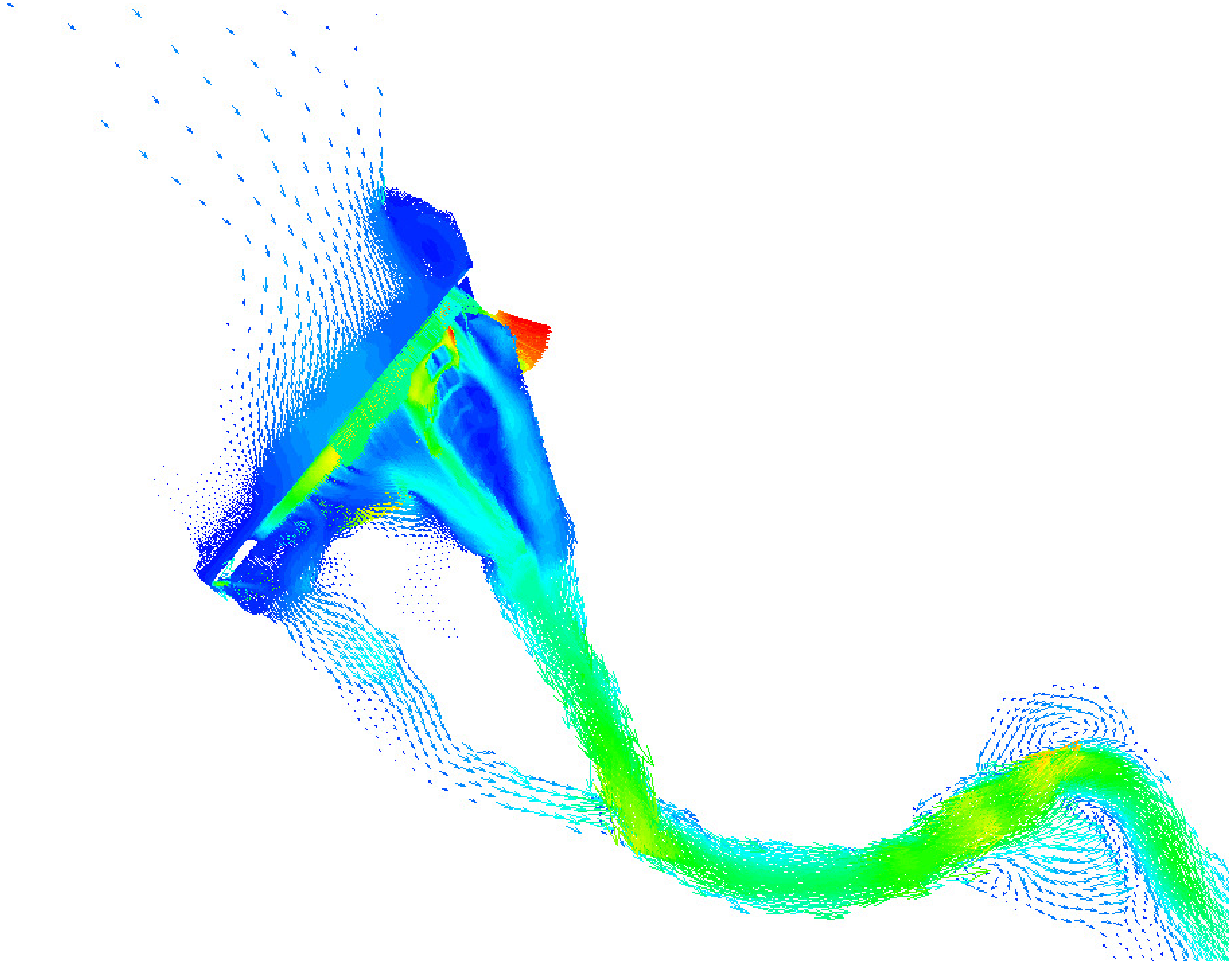
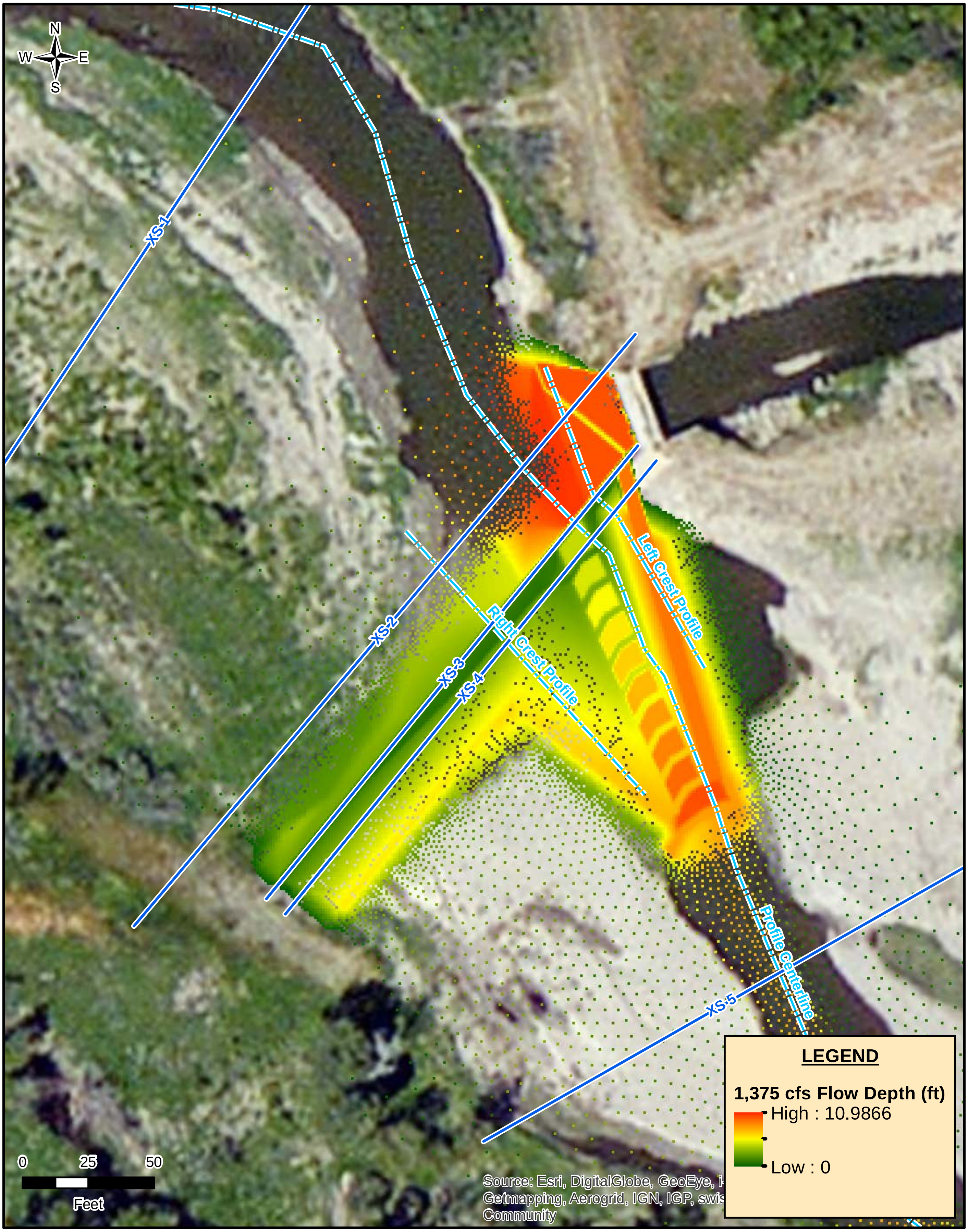
		Post-Project - 500 cfs Flow Velocities	FIGURE NO. A

M:\1447\035\GIS\Exhibits\PP 500cfs Velocity.mxd

Velocity Field







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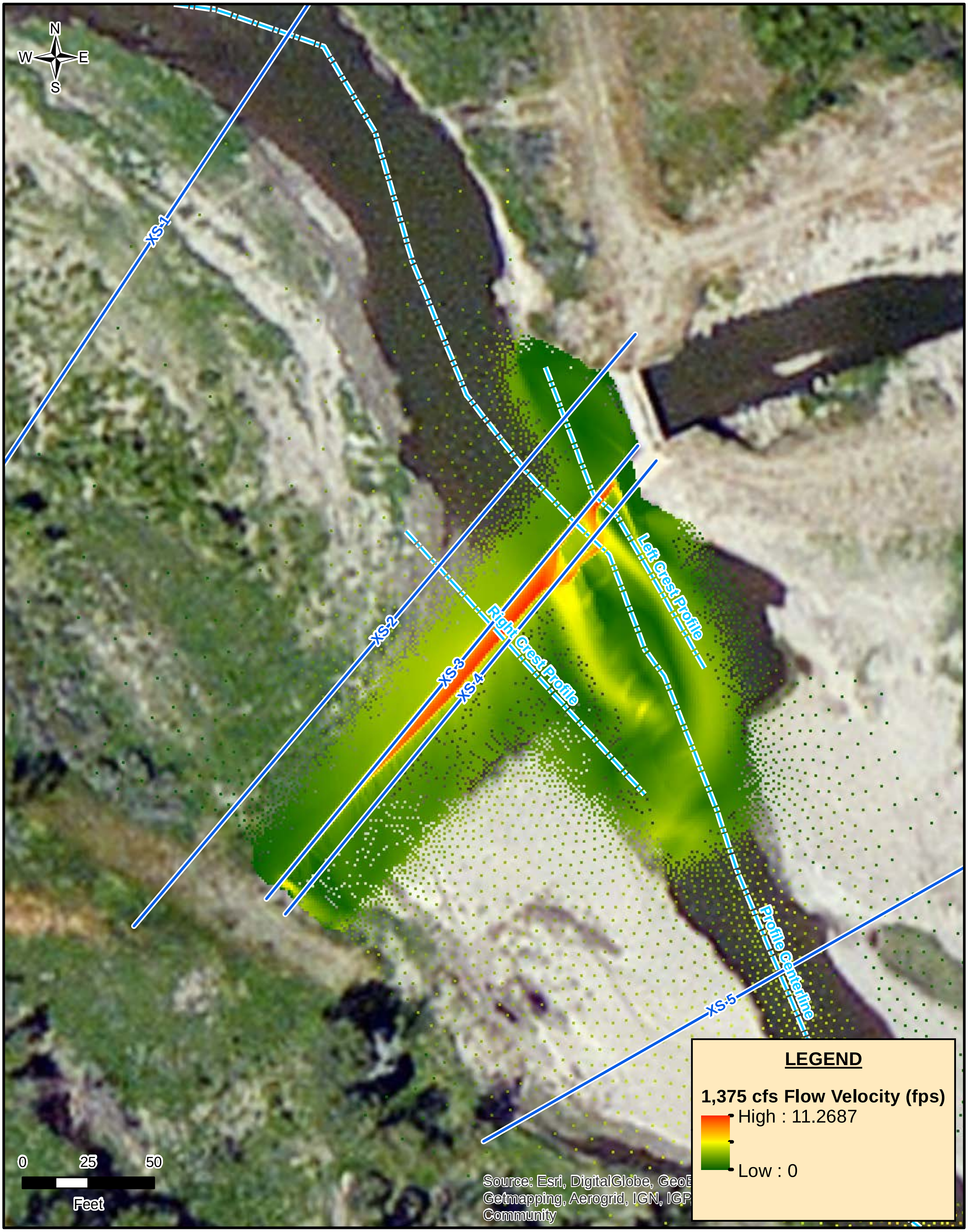
DEADMAN'S BASIN DIVERSION DAM AND HEADGATE REPLACEMENT PROJECT
WHEATLAND COUNTY MONTANA

Post-Project - 1,375 cfs (2-yr) Flow Depth

PROJECT NO. 1447.035

FIGURE NO. A



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DEADMAN'S BASIN DIVERSION DAM AND HEADGATE REPLACEMENT PROJECT

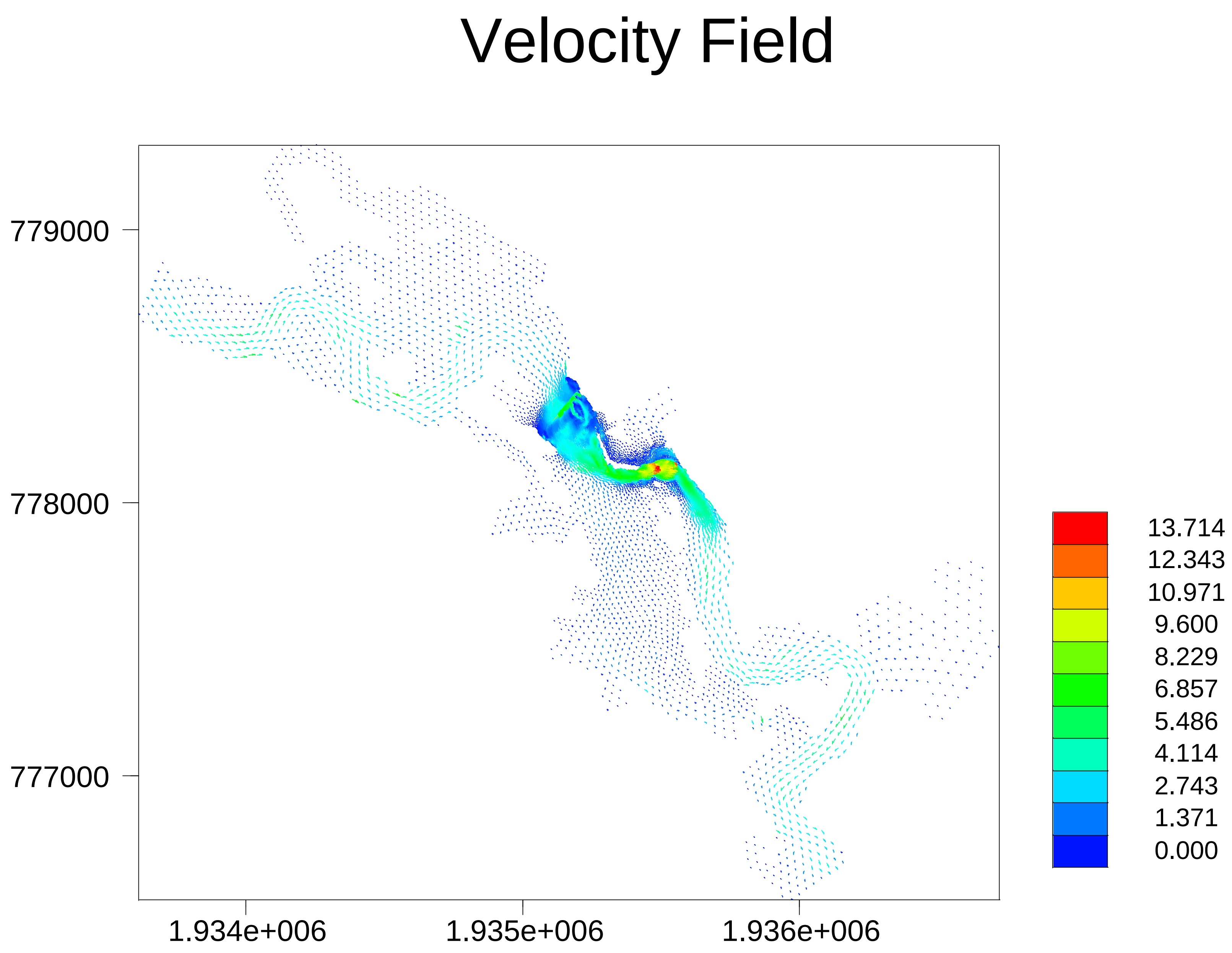
WHEATLAND COUNTY MONTANA

Post-Project - 1,375 cfs (2-yr) Flow Velocities

PROJECT NO. 1447.035

FIGURE NO. A

M:\1447\035\GIS\Exhibits\PP 1375cfs Velocity.mxd


Velocity Field
779000
778000
777000
1.934e+006
1.935e+006
1.936e+006
13.714
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10.971
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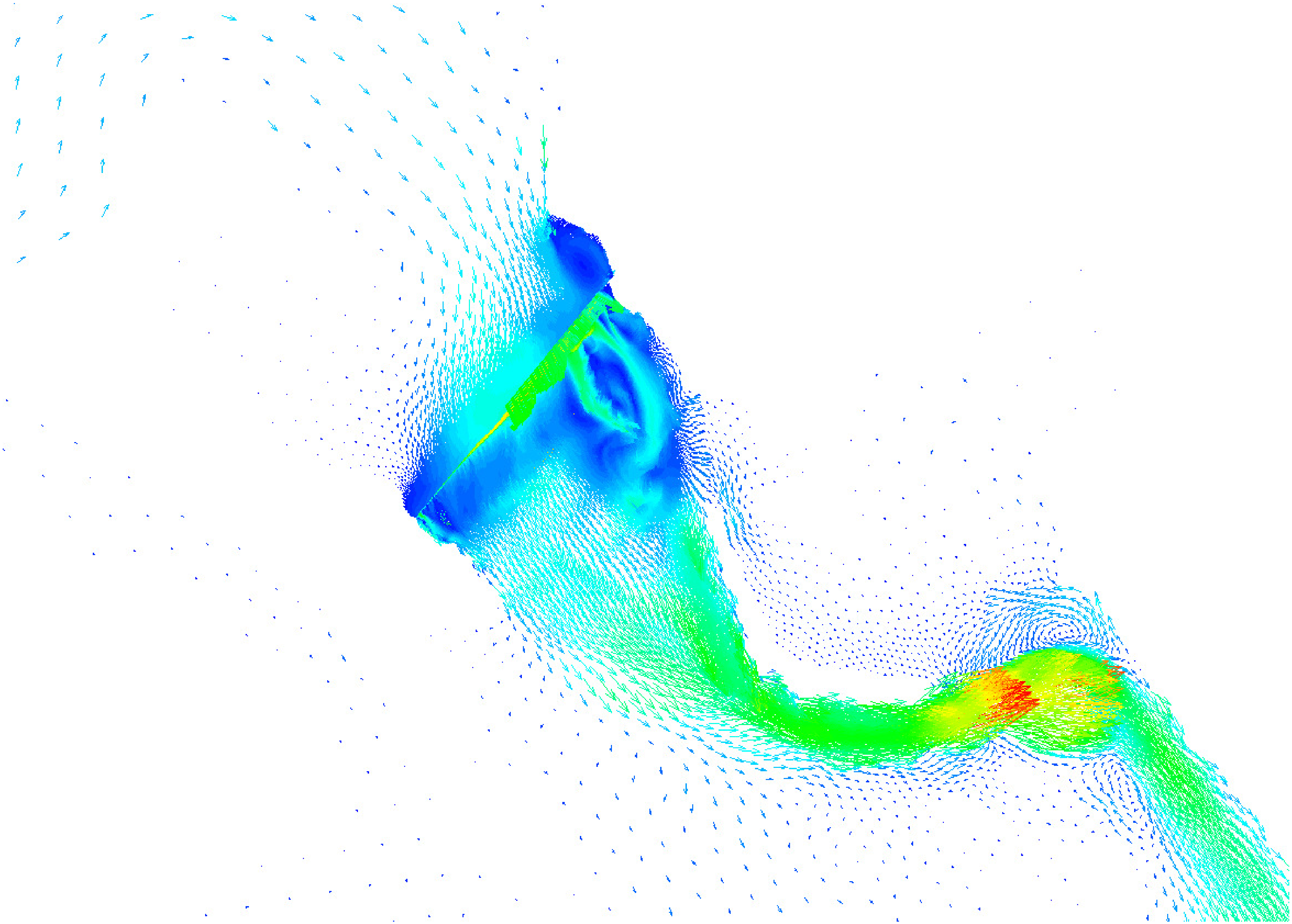
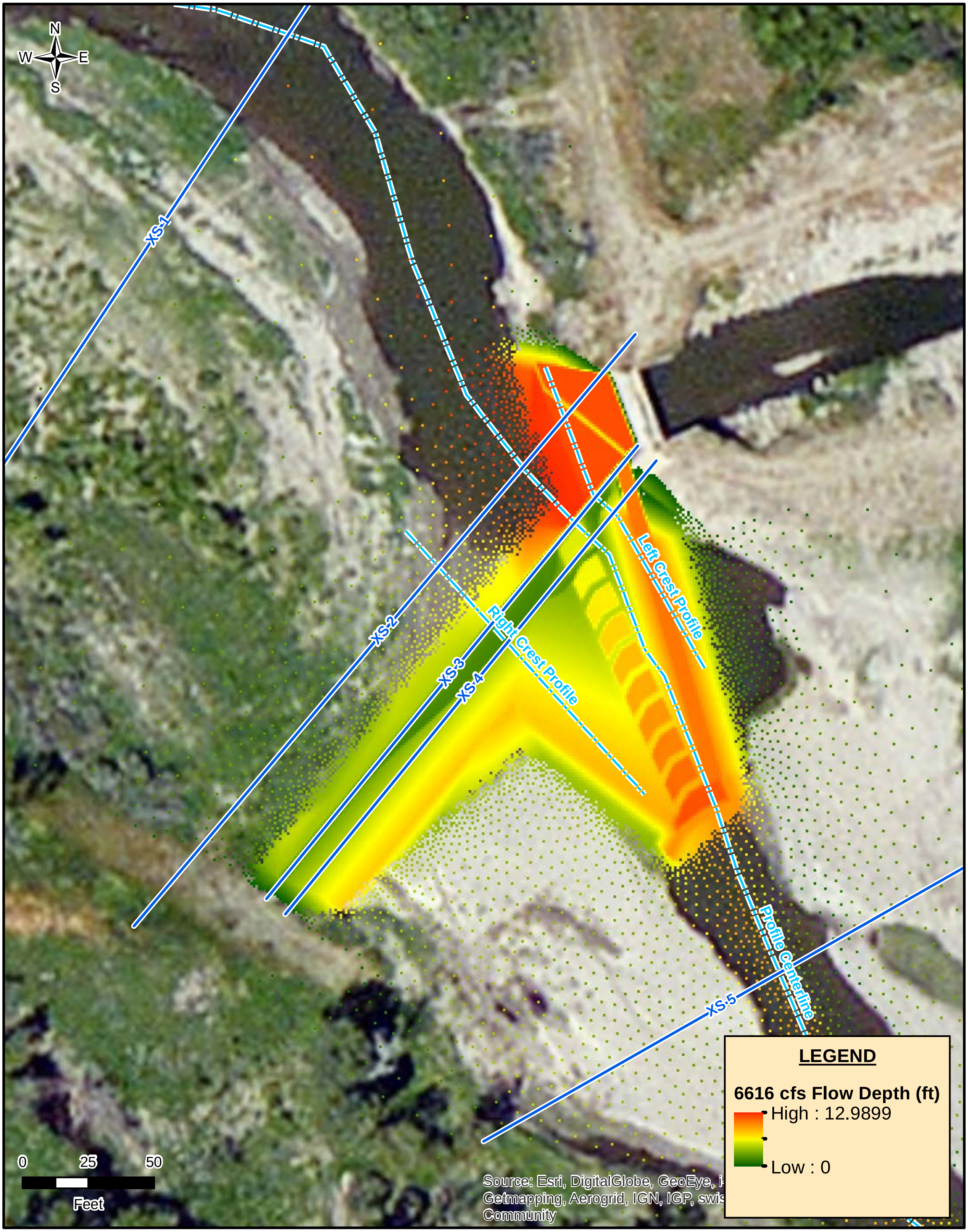
2.743
1.371
0.000





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DEADMAN'S BASIN DIVERSION DAM AND HEADGATE REPLACEMENT PROJECT

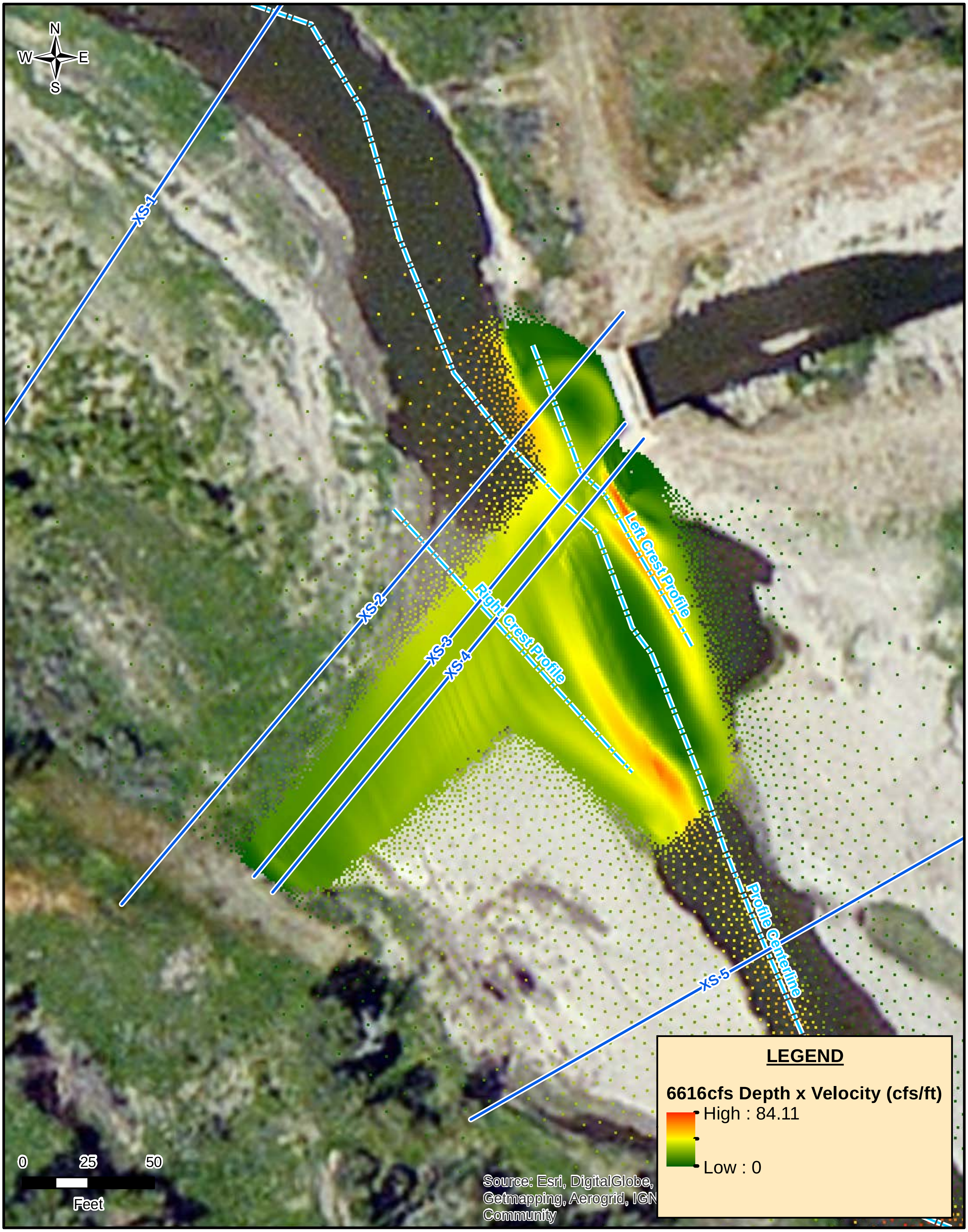
WHEATLAND COUNTY MONTANA

PROJECT NO. 1447.035

Post-Project - 6,616 cfs (100-yr) Flow Depth

FIGURE NO. A

M:\1447\035\GIS\Exhibits\PP 6616cfs Depth.mxd



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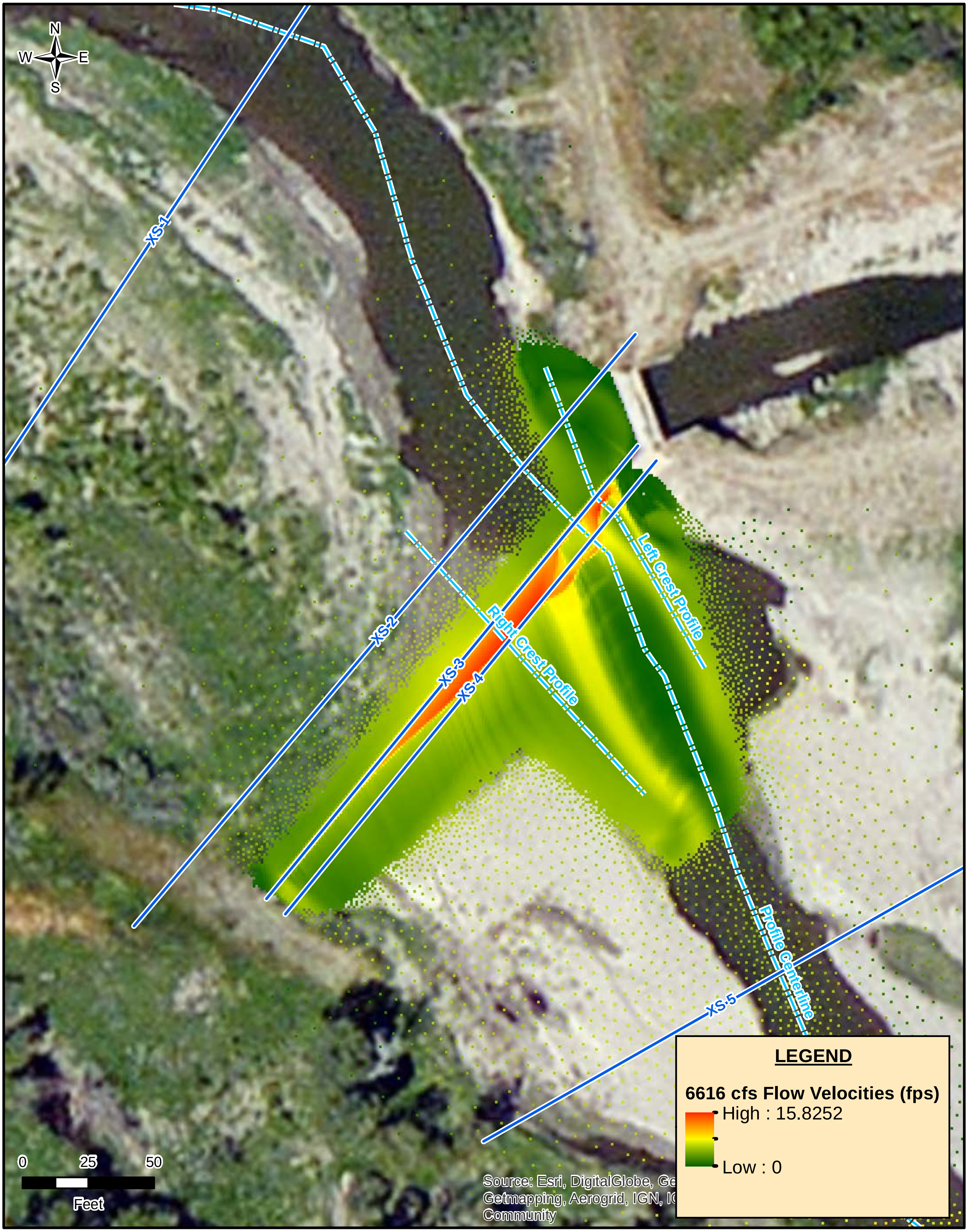
DEADMAN'S BASIN DIVERSION DAM AND HEADGATE REPLACEMENT PROJECT
WHEATLAND COUNTY MONTANA

Post-Project - 6,616 cfs (100-yr)
Depth times Velocity

PROJECT NO.
1447.035

FIGURE NO.
A

M:\1447\035\GIS\Exhibits\PP 6616cfs DepthxVelocity.mxd



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DEADMAN'S BASIN DIVERSION DAM AND HEADGATE REPLACEMENT PROJECT

WHEATLAND COUNTY MONTANA

PROJECT NO. 1447.035

Post-Project - 6,616 cfs (100-yr) Flow Velocities

FIGURE NO. A

Velocity Field